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AMERICAN SOCIETY

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(INSTITUTED 1852)

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VOLUME 97

1933

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Another copy filed Association Transactions
Bridges

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 1818

GEORGE WASHINGTON BRIDGE: GENERAL CONCEPTION AND DEVELOPMENT OF DESIGN

By O. H. Ammann,[1] M. Am. Soc. C. E.

Synopsis

The principal purpose of this paper is to review the developments which after many years of effort have led to the realization of the bridging of the Hudson River at New York, N. Y.; to outline the conditions and considerations which governed the planning of the bridge between Fort Washington and Fort Lee, known as George Washington Bridge; and to record the conception and development of its design in general, the organization, progress, and the cost of its construction.

In view of the magnitude and complexity of the project, this paper is supplemented by a series of papers which deal, more in detail, with the various important phases of the project.

History, Legislation, and Financing

The bridging of the Hudson River at New York is one of those civic and engineering undertakings which for generations have attracted the ambitions and efforts of public-spirited men and of engineers, but the development of which to the point of success must, on account of their magnitude and the many ramifications, consume years and are attained only after a series of unsuccessful attempts.

Credit for the ultimate success, therefore, is due not only to those who are so fortunate as to accomplish the execution, but as much, or more, to those pioneers who by their vision and courage have pointed the way, and whose ideas and studies have prepared the ground for the final accomplishment.

[1] Chf. Engr., The Port of New York Authority, New York, N. Y.

As might be expected, the various attempts to bridge the Hudson, initiated by different interests and involving different conceptions, have produced a wealth of varied ideas regarding the location of crossings and their type and capacity, not to speak of the many divergent opinions regarding questions of design.

While it is to be assumed that the projects which have thus far materialized owe their success to some outstanding merits, it is also undoubtedly true that favorable circumstances combined to effect their realization. The problems involved are so complex and so fraught with ramifications politically, financially, as well as technically, and subject to such rapidly changing conditions that it would be preposterous to assign to any particular project the virtue of, even passing, perfection and completeness.

In the fifty years since 1880, during which concrete efforts have been made to bridge the Hudson River, conditions which would influence any of the major questions involved in this problem, have changed radically.

Traffic in New York City has increased materially in volume and its center of gravity, with that of population and business, has moved steadily north. In the early attempts, the demand for rail traffic was dominant. Crossings for vehicular traffic were scarcely ever considered. Tunnels were thought to be impracticable even for rail transportation. To-day, any bridge across the Hudson River at New York must be viewed primarily as a highway structure, only incidentally accommodating rail traffic, and it is in sharp competition with the tunnel.

More severe demands are made to-day by navigation interests for clearances under a bridge across the Hudson River, and it appears certain that the War Department will permit nothing less than 175 ft. in clear height.

The development of property and the congested conditions on either side of the river also impose such severe limitations upon the building of bridges as to exclude them, practically, for any location in the lower part of Manhattan.

There has also been a marked change in the attitude of the Governments, reflecting public opinion, with respect to the method of financing and carrying out such far-reaching public improvements as crossings over or under the Hudson River.

Joint financing and building by public agencies representing the two States and under State legislation sanctioned by the Federal Government have proved eminently successful and have gained public confidence.

The history of bridging the Hudson River is necessarily interwoven with the broader developments regarding transportation in and around the Port of New York, but in order to give the project dealt with in this paper its proper historic setting, it will suffice to review briefly the various previous attempts.

Projects of the New York and New Jersey Bridge Company and Investigations by the Federal Government.—As early as 1868, the State of New Jersey passed an Act authorizing the New York and New Jersey Bridge Company to build a bridge across the Hudson River at a suitable point north of the southerly line of the Township of Union. The Act permitted a bridge

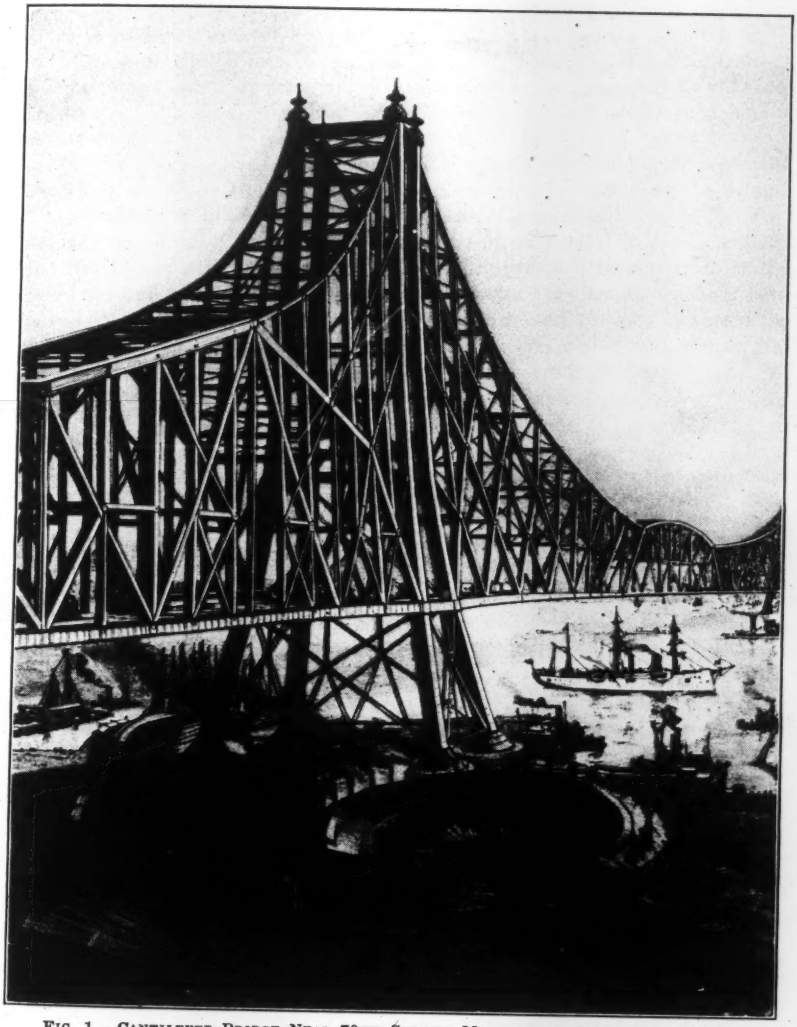


FIG. 1.—CANTILEVER BRIDGE NEAR 70TH STREET, MANHATTAN, PROPOSED IN 1893.

with one or two piers in the river between the bulkhead lines with clear openings of not less than 1 000 ft. and a clear height of 130 ft. at the middle of the river.

In accordance with its provisions the Act did not become effective until, in 1890, New York State passed concurrent legislation giving a charter to a similar company and providing for its consolidation with the New Jersey Company. The New York State Act, however, specifically excluded a pier in the river.

In 1890, these Companies made application to the Federal Government for approval of the State Acts and for permission to construct the bridge. They proposed to build a cantilever structure with a central span of 2 300 ft. between centers of towers, or a clear opening of only 2 000 ft. between piers, the New Jersey pier to be located in the river about 900 ft. riverward of the pier-head line (see Fig. 1). The bridge was tentatively located at about 70th Street, Manhattan, and designed to carry six railroad tracks. It was to be connected by an elevated approach with a union depot near 40th Street and Broadway. The cost of the bridge was estimated at $22 000 000.

In support of this plan, the Bridge Companies[2] argued that,

"A suspension bridge spanning the North River without a pier would involve such elements of uncertainty as regards first cost, novelty in its magnitude as a hitherto untried engineering feat, and time of construction, to say nothing of the well-founded prejudice against the 'suspension' principle for railroad purposes, as would render the enterprise impracticable from a financial standpoint."

On account of much opposition which had arisen against the location of any pier in the river and the controversy relative to the feasibility of a single span across the river, an Act was finally passed and approved by the Federal Government in 1894, authorizing the New York and New Jersey Companies to build a bridge between 59th and 60th Streets, New York City, but provided therein that the plans must be approved by the Secretary of War and also that the President of the United States should appoint a Board, consisting of five competent, disinterested expert bridge engineers, of whom one must be a member of the United States Corps of Engineers, to recommend to the Secretary of War "what length of span, not less than 2 000 ft., would be safe and practicable for a railroad bridge." [2]

Accordingly, a Board composed of the following engineers was appointed by President Cleveland: William H. Burr, M. Am. Soc. C. E. and the late George S. Morison, Past-President, Am. Soc. C. E., and Charles W. Raymond, L. G. L. Bouscaren, and Theodore Cooper, Members, Am. Soc. C. E.

The Secretary of War had previously appointed a Board of Officers of the United States Corps of Engineers with instructions to "investigate and report their conclusions as to the maximum length of span practicable for suspension bridges and consistent with an amount of traffic probably sufficient to warrant the expense of construction." This Board was composed of Col. Edward Burr, U. S. A. (*Retired*) (then Captain, Corps of Engineers,

[2] See Senate Ex. Doc. No. 12, 53d Cong., 3d Session.

U. S. A.), M. Am. Soc. C. E., and the late Brig.-Gen. William H. Bixby, U. S. A. (*Retired*), M. Am. Soc. C. E., (then, also, Captain, Corps of Engineers, U. S. A.), and the late Maj. Charles W. Raymond.

The reports of both these Boards,[2] which have become classic documents, furnished valuable and exhaustive information and definitely disposed of the question of the feasibility of a single span across the Hudson River in New York and the adaptability and economy of the suspension type for long spans. The reports also contain valuable and interesting statements and studies by Gustav Lindenthal, Hon. M. Am. Soc. C. E., and the late Charles Macdonald, Past-President, Am. Soc. C. E., and Wilhelm Hildenbrand, M. Am. Soc. C. E., as well as a theory of the stiffening girder by Professor J. Melan.

The report of the Board of Engineers closed with the following recommendation:

"The only subject referred to your Board is 'to recommend what length of span not less than 2 000 ft. would be safe and practicable for a railroad bridge to be constructed over the Hudson River between Fifty-ninth and Sixty-ninth Streets.' A single span from pier-head to pier-head, built on either the cantilever or suspension principle, would be safe. The estimated cost of the 3 100 foot clear-span cantilever being about twice that of the shorter span, your Board consider themselves justified in pronouncing it impracticable on financial grounds. As the cost of the single span suspension bridge is at most (not more than) one-third greater than that of the 2 000 ft. cantilever, your Board are unable to say that such greater cost is enough to render the suspension bridge impracticable.

"The Board have reached this conclusion after careful study, and they have thought it best to give the full course of reasoning which they have followed. They feel that the contingency attending the construction of the deep-river foundation of the cantilever bridge, even waiving the absolute necessity of carrying this foundation to rock, is enough to balance a part of the greater cost of the suspension bridge.

"The conclusion of this Board is that of a Board of Bridge Engineers acting in a professional capacity. While from such professional view they must pronounce the suspension bridge practicable, they do not in this conclusion give an opinion on the financial practicability and merit of either plan."

The conclusions of the Board of Engineer Officers as endorsed by the then Chief of Engineers, U. S. Army, Gen. Thomas L. Casey, were as follows:

"The final plans for a work of such magnitude would only be adopted after the most extended theoretical and experimental investigations, and the estimated cost would undoubtedly be much reduced by such studies. Assuming the most favorable location and the most competent engineering management, the Board believe that $23 000 000 is a reasonable estimate for a six-track railroad suspension bridge 3 200 feet long, and they consider the amount of traffic which such a bridge would accommodate sufficient to warrant the expense of construction. They believe, however, that the bridge should be so constructed that its capacity can be readily increased, and with the suspension system this can be provided for by giving suitable dimensions to the towers and anchorages."

Briefly, the conclusions are to the effect that both the 2 000-ft. cantilever bridge, which requires a pier in the river and the 3 200-ft. suspension bridge without a center pier, are safe and not impracticable as to cost, and

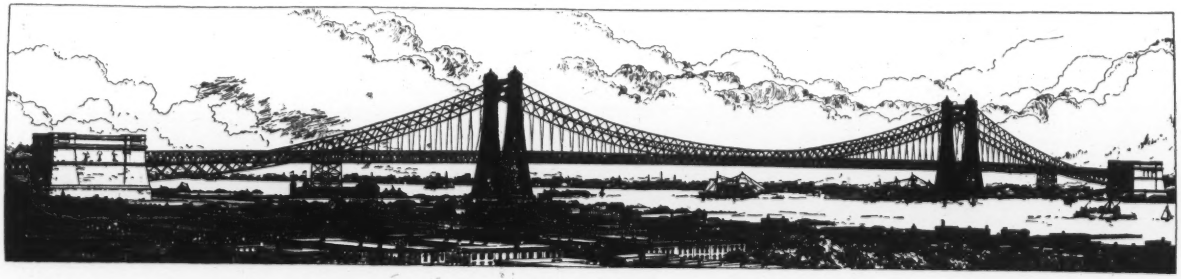


FIG. 2.—SUSPENSION BRIDGE NEAR 23D STREET, MANHATTAN, DESIGNED BY G. LINDENTHAL, IN 1899.

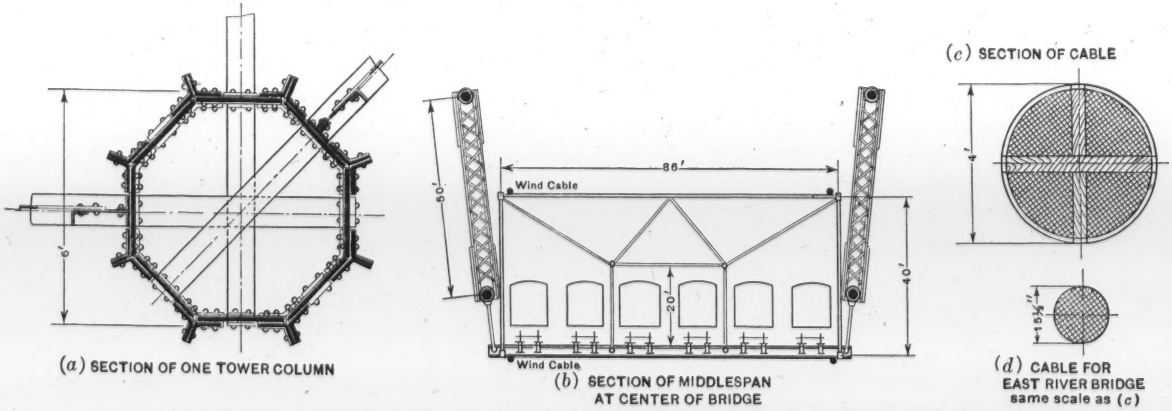


FIG. 3.—SECTIONS FOR SUSPENSION BRIDGE NEAR 23D STREET, MANHATTAN, DESIGNED IN 1899.

that the latter type, in spite of its greater span, would not cost materially more. The estimates of various studies ranged between about $25 000 000 and $35 000 000, depending upon location and capacity.

As a result of these findings, the Secretary of War disapproved of the proposed cantilever structure with a pier in the river.

In the meantime the New York and New Jersey Bridge Company had changed its plans and thereafter proposed a six-track suspension bridge in the vicinity of 57th Street with a single river span at an estimated cost of $25 000 000 for the bridge proper. A number of wash borings were made for the Company by the late Charles B. Brush M. Am. Soc. C. E., at both the 71st Street and the 59th Street locations, but no construction work was ever undertaken.

The Project of the North River Bridge Company.—At a meeting of the Society in January, 1888, Mr. Lindenthal outlined his carefully studied plan for a railroad suspension bridge across the Hudson River. It was a remarkably bold and well conceived plan, calling for the first time for a single span across the river, 2 850 ft. in length, and two side spans of 1 500 ft. each, or a total length between anchorages of 5 850 ft. (Fig. 2). It provided for six railroad tracks to be carried by four 48-in. cables, braced in pairs to form rigid suspended trusses, slightly cradled (Fig. 3). The cables were to be suspended from two pairs of octagonal-shaped steel towers, 525 ft. high, of massive proportions. Subsequently, the plan was modified to provide for as many as fourteen tracks and a promenade, and the span was increased to 3 100 ft.[3]

The bridge proper was estimated to cost $16 000 000 and, with terminal facilities, $40 000 000. A location in the vicinity of 10th Street, New York, was at first selected, but this was later changed to the vicinity of 23d Street, opposite Hoboken, N. J.

The publication of the plans caused widespread and favorable comment, and, in 1890, the Federal Government granted a charter to the North River Bridge Company, by passing an Act authorizing Mr. Lindenthal and his associates (among whom were such other well-known engineers as Samuel Rea, Hon. M. Am. Soc. C. E., Henry Flad, M. Am. Soc. C. E., and Mr. F. W. Roebling, none of whom is now living), to construct a bridge "for at least six railroad tracks, with capacity for four additional tracks for future enlargement, and with a single span across the river between pier-head lines." The charter also provided for the building of the necessary approaches and terminal facilities. The plans for the bridge were approved by the Secretary of War, in December, 1891, who fixed the clear height at the center at 150 ft.

An attempt to build the bridge at that time failed, due evidently to the financial stringency in 1893 and the consequent inability of the railroad companies to co-operate, this being essential for the success of a railroad terminal improvement of such magnitude.

Another effort was made in 1900, when the Pennsylvania Railroad Company invited the other railroads terminating on the west shore of the Hudson River to join in the building of the bridge; but these companies did not avail

[3] *Scientific American*, May 23, 1891, p. 319.

themselves of the opportunity and the Pennsylvania Railroad Company, having become convinced of the feasibility of electric railroad traction, decided to enter Manhattan by two single-track tunnels near 33d Street.[4]

Furthermore, having been encouraged by the example of the Pennsylvania Railroad, and in co-operation with the latter, the Hudson and Manhattan Railroad Company, in 1910, completed its two pairs of tubes connecting three of the passenger stations on the New Jersey side with a down-town terminal and with points along Sixth Avenue, as far north as 33d Street.

The successful completion of these tubes for electric rail traffic constituted a setback to the possibilities of a railroad bridge in that they established beyond doubt the feasibility of tunnels for rail traffic and obviated the necessity for additional crossings for the time being.

The phenomenal growth of vehicular traffic after the World War and the renewed efforts, more particularly on the part of the New York and New Jersey Port and Harbor Development Commission (later succeeded by the Port of New York Authority), to improve rail terminal facilities in New York, gave new encouragement to the sponsors of a bridge to revive the North River Bridge Company's plans.

The plans for both the bridge proper and the terminal facilities were completely revised by Mr. Lindenthal. The location had previously, upon permis-

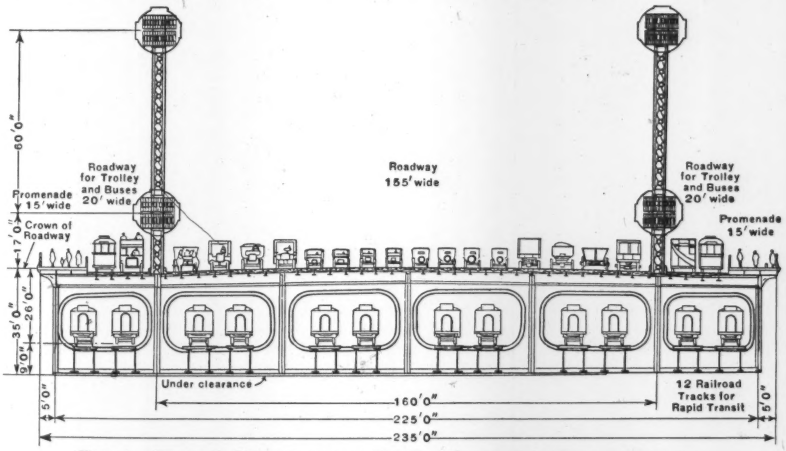


FIG. 4.—TYPICAL CROSS-SECTION; EYE-BAR SUSPENSION BRIDGE DESIGN, SUBMITTED IN 1923.

sion by the Federal Government, been removed to the vicinity of 57th Street, Manhattan. The bridge proper was re-designed for a capacity of twelve railroad and rapid transit tracks, twenty lanes of vehicular and bus traffic, and two 15-ft. promenades, with a total width of the double deck floor of 235 ft. (Fig. 4). The suspended system with a central span of 3 240 ft. remained essentially the same, except in its proportions and the substitution of eye-bar chains in the place of wire cables. The towers were designed as rigid steel

[4] *Transactions*, Am. Soc. C. E., Vols. LXVIII and LXIX (1910).

frames to be enveloped by an independent shell of masonry of massive appearance. The bridge proper and highway approaches were estimated to cost approximately $180 000 000.

With the assistance of Francis Lee Stuart, Past-President, Am. Soc. C. E., and in consultation with the engineers of the various railroad companies, Mr. Lindenthal developed an elaborate plan for freight and passenger terminal facilities. The plan was submitted to the Port of New York Authority in 1921, but the latter, while recognizing the merits of the bridge as a highway proposition, could not see its way clear to adopt it as a feature of its comprehensive plan in the solution of the freight terminal problem.[5]

In 1923, the North River Bridge Company submitted the revised plans to the War Department, proposing a clear height under the bridge at the center of 175 ft. After lengthy consideration, the War Department on June 9, 1931, rendered a decision that the clear height should not be less than 200 ft. at the center and 185 ft. at the pier-head lines.

The Interstate Bridge and Tunnel Commissions of New York and New Jersey and the Holland Vehicular Tunnel.—In 1906, the Governors of New York and New Jersey, acting under laws passed in that year by the respective Legislatures, appointed Commissions, known as the Interstate Bridge Commissions, for the purpose of considering the construction of one or more bridges over the Hudson River at the joint expense of the two States. These Commissions collaborated in making a careful study of various possible sites and in 1909 and 1910, reported favorably upon the bridge located at 179th Street, opposite Fort Lee. The report of the New York Commission of 1910 contained the following recommendations:

"From the purely engineering point of view it is the most economical crossing from Manhattan over the Hudson River that it is possible to select, it being the narrowest part of the river, with comparatively small land damages on either side. The approaches over land are short, that from New York reaching 179th Street over Fort Washington Park, and that from New Jersey over the proposed limits of Palisade Park. The foundation problems are not likely to be of great magnitude as far as can be judged in the absence of borings. The rock is on the surface at Fort Washington point, involving no foundation work whatever, beyond levelling off the same. Further, the channel span need not, in our engineer's opinion, be over 1 400 feet or thereabouts, which will give abundant passage for all river traffic, the north limit anchorage for large vessels being below this crossing. This site has not been bored, but in our engineer's opinion, from the apparent geological condition, 10 million dollars will cover the cost of a bridge at this point for highway and speed trolley service, being in their opinion one-third the cost of a bridge lower down the river."

Subsequently, borings were undertaken at the 57th, 110th, and 179th Street locations. The assumption that piers could be placed in the river at the 179th Street site, and that the foundation problems would not be of great magnitude, was not supported by the results of these borings and, with the prospect that a bridge at that point would require a single span across the river, as elsewhere, the engineers of the Commissions, the late A. P. Boller and Henry W. Hodge, Members, Am. Soc. C. E., reported in 1911, as follows:

[5] See statement by Eugenius H. Outerbridge, Chairman, Port of New York Authority, to the Advisory Council, December 8, 1921.

"The borings conclusively prove that there are no practicable foundation conditions outside of the pier head lines, forcing the inevitable conclusions that any bridge contemplated over the Hudson River within the limits of the City of New York must have at least a single span over the river between the pier head lines. Inasmuch as the distance between pier head lines is substantially the same at any proposed crossing between 57th Street and 179th Street, the constructive cost of a bridge at any site will practically be the same; such being the case your engineer is of the opinion that the final determination of the bridge location should be guided by the line of greatest natural travel and public convenience. While the cost of real estate and abutting damages will vary according to location, still it is believed that the needs of the community served should be controlling within reasonable limits. All things considered, it is the firm opinion of your engineer that a bridge located in the neighborhood of 59th Street will best conform to the needs of population density and requirements on both sides of the river, and such location is recommended for adoption."

Thereafter, the plan for a bridge at 179th Street seems to have been abandoned by the Interstate Bridge Commissions, and, in 1913, they recommended a bridge near 59th Street, a design for which had been prepared by Messrs. Boller and Hodge, and H. C. Baird, M. Am. Soc. C. E. At the same time, the Commissions reported favorably upon a vehicular tunnel at Canal Street, which latter project had been investigated and recommended by Messrs. Jacobs and Davies, Consulting Engineers, as being feasible and economical.

It is quite evident that by this time the need for more adequate crossings for vehicular traffic had come to the foreground, and furnished new possibilities for bridging and tunneling the Hudson River.

The beginning of the World War delayed the undertaking of either project, and only in 1919, when the necessity for a crossing became urgent as a result of the rapidly growing vehicular traffic and activities arising out of the war, the States of New York and New Jersey entered into a treaty for the construction of a vehicular tunnel by and through the New York Bridge and Tunnel Commission and the New Jersey Interstate Bridge and Tunnel Commission (successors of the aforementioned Bridge Commissions), the outcome of which was the successful completion, in 1927, of the Holland Tunnel between Canal Street, Manhattan, and Jersey City, N. J.

Preference to the tunnel project over the bridge at 57th Street was evidently given partly on account of the location of the tunnel down town, where a crossing could be of more immediate relief to vehicular traffic; and partly due to its apparent lower cost, the tunnel having been estimated in 1913 at $11 000 000 and the bridge at 57th Street at $42 000 000.

The design of the bridge by Messrs. Boller, Hodge, and Baird contemplated a capacity of a single deck, 204 ft. wide, for eight rapid transit and trolley tracks, two roadways, each 36 ft. wide, and two sidewalks, each 8 ft. wide. The suspended truss type of bridge was selected, each of four trusses consisting of a main eye-bar cable stiffened by a secondary cable and the connecting web members. The central span was assumed at 2 880 ft. center to center of towers and the clear height over the river at 170 ft.

The Port of New York Authority and the Financing of the George Washington Bridge.—Realizing the urgent need for additional crossings between the two States, as brought about by the phenomenal growth of vehicular traffic, Governor Alfred E. Smith, of New York State, and Governor George S. Silzer, of New Jersey, on August 5, 1923, issued a joint proclamation, in which they stated, in part:

"One of the results of the conference between the two Governors was that we favor the construction at the earliest possible moment of additional vehicular tunnels or bridges between the State of New York and the State of New Jersey to be determined upon, constructed and financed by the Port of New York Authority, and we stand ready to recommend to the Legislatures the passage of any additional legislation that will be helpful towards the accomplishment of this result."

On December 5, 1923, a public hearing on this subject was held by the Port Authority. The sentiment expressed at this hearing was almost unanimous in favor of the building of interstate vehicular tunnels or bridges by the Port of New York Authority and, likewise, in favor of two or more vehicular tunnels, and a bridge at some point north of 128th Street, Manhattan. There was substantial approval for a highway bridge at a location about 178th Street, Manhattan.

In its report[6] to the Governors of the two States in December, 1923, the Port Authority recommended that preliminary engineering and traffic studies and plans should be promptly undertaken relating to such crossings.

Substantial weight to the proposal for a bridge at 179th Street was given by a report to the Port Authority of the Committee on Plan of New York and Its Environs of the Russell Sage Foundation. In a subsequent publication[7] that Committee outlined a plan for the bridge in which a central span of 2 700 ft., with a pier approximately 400 ft. beyond the westerly pier-head line, was tentatively assumed.

In a communication to the Port Authority in December, 1923, Governor Silzer, of New Jersey, transmitted to that body for its consideration the study of a plan for a bridge at 179th Street which had been submitted to him by the writer and which, based upon a carefully studied design (Fig. 5), and estimates of cost and revenue, indicated the financial feasibility of the project. In its essential features the writer's tentative plan substantially agrees with the design eventually adopted for execution. It provided for a single span of 3 400 ft. across the river, with piers back of the pier-head lines, a capacity for eight lanes of vehicular traffic, two sidewalks, and four rapid transit tracks, and a clear height of 200 ft. above the water. (See Fig. 6.) The plan was presented by the writer at the Annual Meeting of the Connecticut Society of Civil Engineers on February 19, 1924.[8]

In 1925, the States of New York and New Jersey passed legislation authorizing and empowering the Port of New York Authority, in partial effectuation of the comprehensive plan for the development of the Port of

[6] Annual Rept. of The Port of New York Authority, January, 1924, p. 43.

[7] "Some Preliminary Suggestions for the Relief of Highway Congestion in New York," 1925.

[8] *Proceedings*, Connecticut Soc. of Civ. Engrs., 1924.

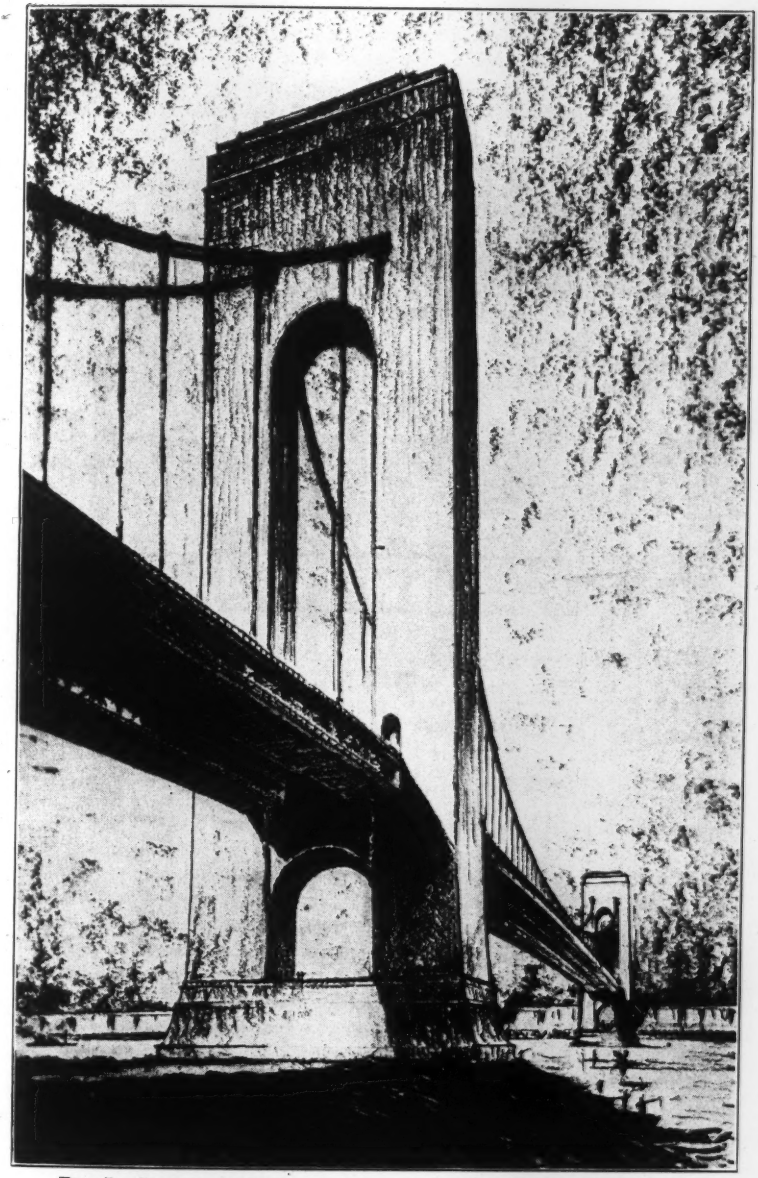


FIG. 5.—SUSPENSION BRIDGE PROPOSED FOR 179TH STREET, MANHATTAN, IN DECEMBER, 1923.

New York, to construct, operate, and maintain a bridge across the Hudson River, from points between 170th Street and 185th Street, Manhattan, and points approximately opposite thereto in Fort Lee, N. J.

In passing this legislation, the Legislatures of the two States fully recognized the fact, as stated in the Acts,[9] that,

"The construction, maintenance and operation of said bridge is in all respects for the benefit of the people of the two States, for the increase of their commerce and prosperity, and for the improvement of their health and living conditions, and the Port Authority shall be regarded as performing a governmental function in undertaking the said construction, maintenance, and operation and in carrying out the provisions of law relating to the said bridge and shall be required to pay no taxes or assessments upon any of the property acquired by it for the construction, operation, and maintenance of such bridge."

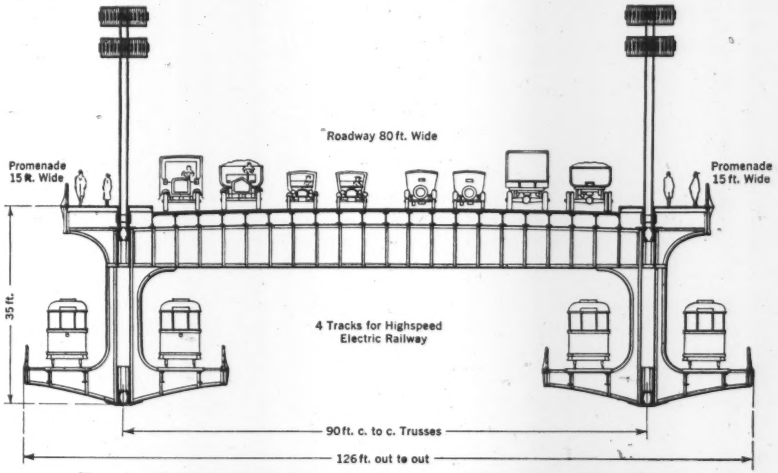

Roadway 80 ft. Wide
Promenade 15 ft. Wide
Promenade 15 ft. Wide
35 ft.
4 Tracks for Highspeed Electric Railway
90 ft. c. to c. Trusses
126 ft. out to out

FIG. 6.—TYPICAL CROSS-SECTION, PROPOSED DESIGN FOR 179TH STREET SUSPENSION BRIDGE

The Acts carried an appropriation of $100 000 from each State to enable the Port Authority to undertake the preliminary surveys and studies. In the same year, Congress also passed an Act authorizing the Port Authority to build this bridge, subject to the approval of the War Department.

Work on preliminary studies was started in July 1925. A tentative report on the physical and financial feasibility of the undertaking was sent to the Governors of the two States in February, 1926, and thereupon the two States enacted further legislation pledging to the Port Authority, in aid of the prompt and economical construction of the bridge, the sum of $5 000 000 from each State. The Legislatures also voted an additional appropriation of $50 000 each to permit the completion of the preliminary work.

The statutes provide that moneys advanced by the States, together with interest at the rate of 4%, are to be repaid eventually by the Port Authority

[9] Port Authority Statutes.

from the revenues and tolls arising out of the use of the bridge. The remainder of the funds required for the construction is to be raised by the Port Authority on its own securities.

Having early in the same year established the practicability and success of this method of financing in connection with the two interstate bridges across the Arthur Kill, the Port Authority in December, 1926, authorized for the Hudson River Bridge an issue of $60 000 000, Port of New York Authority, New York and New Jersey Interstate Bridge Gold Bonds, $20 000 000 of which were sold to a group of underwriters headed by the National City Company of New York. The 4% bonds were sold to the public on the basis of an interest yield of 4.2 per cent. They are secured by a first lien in the revenues which will be derived from the tolls and are to be amortized out of a sinking fund from the revenues within 25 to 30 years. An additional issue of 4½% bonds was sold by the Port Authority in October, 1929, when construction was well under way.

The plans for the George Washington Bridge were submitted to the War Department in December, 1926, and, after a public hearing conducted by Col. R. R. Ralston, U. S. District Engineer, they were promptly approved by the Chief of Engineers, Major General Edgar Jadwin, M. Am. Soc. C. E., and for the War Department by the Hon. Hanford MacNider, Assistant Secretary of War.

Geographical and Traffic Situation, and Economic Justification of the George Washington Bridge

While the State Acts specified the general location of the bridge, the Port Authority considered it essential to determine by careful investigation whether a crossing in that locality was needed and economically justified. The State Acts, moreover, left entire freedom in the determination of the kind and volume of traffic which the bridge was to accommodate. These questions could be answered only on the basis of a thorough survey of existing and prospective traffic conditions.

In reviewing the economic and traffic situation it must be kept in mind, in view of earlier conclusions reached relative to the most advantageous location of a bridge, that, within the past few decades, large centers of population have grown up in the northern part of Manhattan, the Borough of The Bronx, and Westchester County, while on the opposite side of the Hudson River there still remain comparatively undeveloped areas which strongly attract an overflow of population from the congested centers. The mere prospect of the coming of the bridge stimulated tremendous activities in the development of those areas years in advance of its completion. There are in Northern New Jersey also important industrial centers (Fig. 7), as Paterson, Passaic, Hackensack, etc., which have developed a rapidly growing volume of commercial traffic to and from New York City.

If the developments may be taken as an indication of social and economic demands of the population, and if the resulting appreciation in value of real estate alone in the territory contiguous to the bridge, even to date (1932), may be taken as a measure of economic benefit to the people, then the bridge has

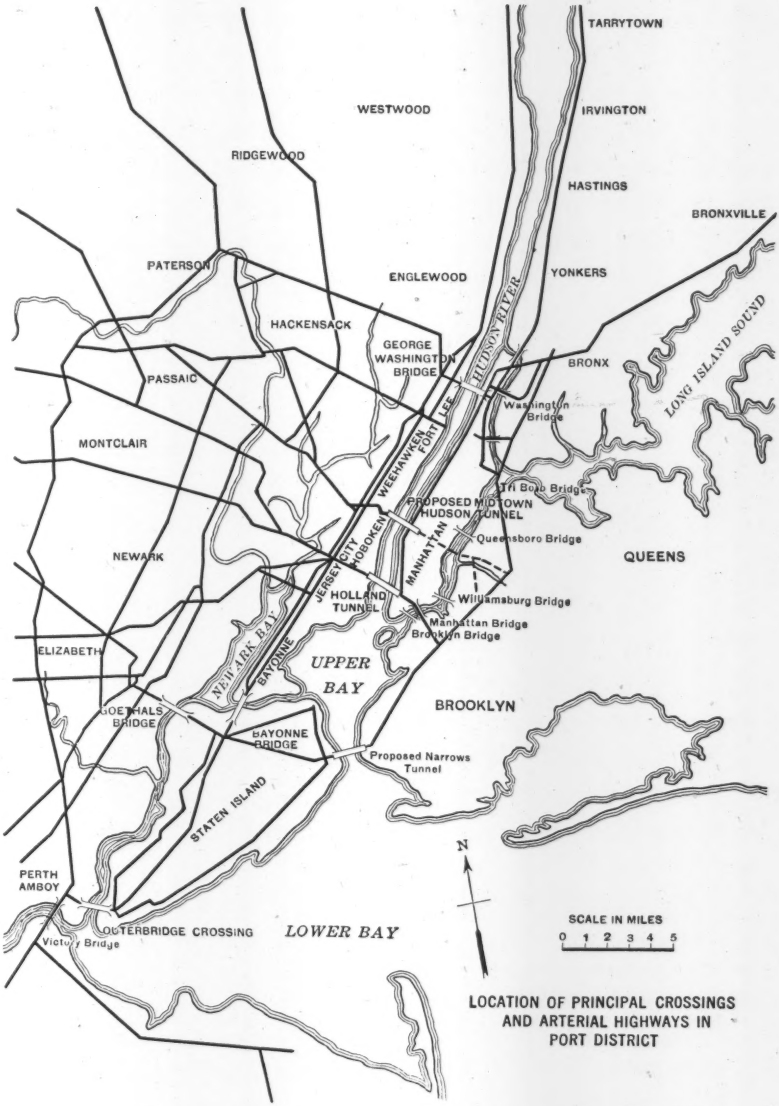


FIG. 7.—PRINCIPAL CROSSINGS AND ARTERIAL HIGHWAYS IN THE PORT OF AUTHORITY DISTRICT.

already more than paid for itself and has demonstrated, in the broadest sense, the economic justification of its construction. Not only that, but this rapid accession in values, although largely stimulated by the wave of prosperity (1922–1929) plainly indicates the need and justification for additional crossings within the Metropolitan District.

Of course, the Port Authority could not depend on speculative or even assured increase in property values or other benefits to the people; its justification for the construction and investment of the capital had to be based solely upon the prospective revenue from reasonable toll charges.

In passing upon the justification of the location of a crossing so far up town, it must also be considered that the most urgent need for more adequate means of crossing the river in the down-town section of Manhattan was at that time in prospect of being met there by the impending completion in 1927 of the Holland Tunnel.

The traffic studies of the Port Authority and others revealed indeed the urgent necessity and justification of a crossing between Upper Manhattan and Northern New Jersey to take care of the rapidly growing vehicular traffic across the river in that vicinity and to relieve the situation growing out of the inadequacy of ferry transportation. The anticipated traffic developments have since been realized beyond expectation, as will be seen from the recorded increase in volume of trans-Hudson traffic.

Besides meeting local traffic demands, it was recognized that the bridge would form an important link in the arterial highway system in the two States and beyond. While the bridge accommodates principally traffic between Northern New Jersey and New York State west of the Hudson and New York City, it also serves traffic from Southern New Jersey, the Atlantic seaboard, and Eastern Pennsylvania, to points in New York State east of the river, New England, and Canada. This long-distance traffic thus avoids passing through the most congested parts of New York City.

In conjunction with the Washington Bridge across the Harlem River and probable future crossings over that river, and the Tri-Borough Bridge across the East River, the George Washington Bridge establishes an uninterrupted highway artery between Northern New Jersey and Long Island.

The traffic studies indicated clearly that the revenue from vehicular traffic alone would be sufficient to cover operating charges, interest, and amortization, of a bridge designed for a capacity to accommodate vehicular as well as rail-passenger traffic far in excess of that required for many years after the opening of the bridge to traffic.

Estimates of traffic across the Hudson River were made by the Port Authority Staff for each year from 1924 to 1960, a period of thirty-seven years. These estimates were based upon actual counts of the number and type of vehicles and their origin and destination during the average and peak months of traffic on the seventeen Hudson River ferries between the Battery, New York City, and Tarrytown, N. Y., in 1925.

The Holland Tunnel was opened to traffic on November 13, 1927. Its probable effect had to be taken into consideration in estimating the traffic across the George Washington Bridge. On the other hand, due allowance was justified for traffic that would be generated by the development of the terri-

tory contiguous to the bridge and of Bergen County generally. Careful records were kept of the Hudson River traffic in subsequent years, and these were supplemented by additional counts in 1929, and a complete review of the distribution of the Hudson River traffic in 1930 in connection with studies for the proposed Midtown Hudson Tunnel in the vicinity of 38th Street, Manhattan.

Table 1 gives, for a number of years, the volume of vehicular traffic across the Hudson River from the Battery, in New York City, to Tarrytown, and the volume that would be diverted to the bridge, as forecast in 1926 at the time when the first bond issue was sold.

In comparison, the recorded actual volume of trans-Hudson vehicular traffic is shown for a number of years from 1920 to 1930. It is significant that, despite the general economic depression which set in in 1929 and continued through 1930, this traffic increased about 25% within those two years and nearly 100% within the five years from 1925 to 1930, or at a rate almost twice as fast as that forecast in 1926. The increase was most rapid at the crossings at and north of 42d Street, the region which the George Washington Bridge will serve most directly. Furthermore, the increase in population and motor-vehicle registration has been particularly rapid in the territories lying nearest the bridge. Population increased 73% in The Bronx, 82% in Westchester County, and 82% in Bergen County, during the ten-year period from 1920 to 1930, as compared to the 29% total increase for the Metropolitan District. In the 5-year period between 1924 and 1929, motor-vehicle registrations for

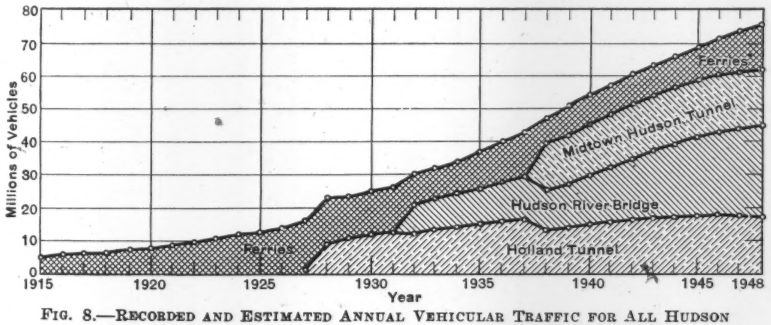


FIG. 8.—RECORDED AND ESTIMATED ANNUAL VEHICULAR TRAFFIC FOR ALL HUDSON RIVER CROSSINGS.

The Bronx, Westchester County, and Bergen County increased 95%, 87%, and 172%, respectively, as compared to 77% for the Metropolitan District as a whole.

In view of these developments—and even taking into account the effect which the proposed Midtown Hudson Tunnel in the vicinity of 38th Street, Manhattan, would have upon the flow of traffic over the George Washington Bridge (see Fig. 8)—it may be confidently expected that the traffic across the latter will exceed the figures forecast in 1926. This is reflected in the 1930 forecast of traffic, across the Hudson River and across the bridge as shown in the last two columns of Table 1.

TABLE 1—Recorded and Estimated Vehicular Traffic Across Hudson River from the Battery, New York City, to Tarrytown, N. Y., and Estimated Traffic Across George Washington Bridge.

Year	1926 forecast of number of vehicles across Hudson River	1926 forecast of number of vehicles to cross on George Washington Bridge	Recorded number of vehicles across Hudson River to 1930	1930 forecast of vehicles across Hudson River, considering effect of opening of Midtown Hudson Tunnel, in 1937	1930 forecast of number of vehicles to cross on George Washington Bridge considering effect of Midtown Hudson Tunnel
1920.....	.....	.....	7 660 000	.....	.....
1924.....	.....	.....	11 890 000	.....	.....
1925.....	.....	.....	12 570 000	.....	.....
1926.....	14 200 000	.....	13 740 000	.....	.....
1927.....	15 500 000	.....	16 090 000	.....	.....
1928.....	16 900 000	.....	20 720 000	.....	.....
1929.....	18 300 000	.....	23 550 000	.....	.....
1930.....	19 800 000	.....	25 500 000	.....	.....
1932.....	22 700 000	8 600 000	.....	30 100 000	8 700 000
1934.....	25 600 000	9 800 000	.....	34 200 000	10 100 000
1938.....	30 600 000	12 000 000	.....	47 000 000	11 500 000
1942.....	34 500 000	13 500 000	.....	60 400 000	18 600 000
1946.....	37 400 000	15 000 000	.....	71 000 000	25 100 000

Based upon the 1926 forecast of traffic and an average toll rate of 50 cents per vehicle (which is a fair approximation of the average rates charged on the ferries in the vicinity of the bridge), and 5 cents per passenger in vehicles, exclusive of driver, the annual net revenue was estimated to increase from a minimum of $5 250 000 in 1932, the first year of operation, and to be sufficient with a substantial surplus to meet interest and sinking-fund payments, during a period of amortization of about twenty-five years, as well as refund with interest to the two States of the amounts advanced by the latter.

The toll charges adopted by the Port Authority are, as follows:

Vehicle	Rate
Motorcycles; bicycles	$0.25
Passenger automobiles; horse-drawn vehicles (2 axles)	0.50
Passenger automobiles and two-wheel trailers (3 axles)	0.70
Trucks up to and including 2 tons capacity (2 axles)	0.50
Trucks of more than 2 tons and including 5 tons capacity (2 axles)	0.75
Trucks of more than 5 tons capacity (2 axles)	1.00
Tractor and trailer; truck, 6 wheels (6 wheels and 3 axles)	1.25
Tractor and trailer; truck and trailer (8 wheels and 4 axles)	1.50
Buses, 4 wheels	1.00
Buses, 6 wheels	1.10
Pedestrians	0.10

Traffic Capacity of Bridge.—Based on the estimated traffic figures and a capacity per lane per peak hour of 1 400 vehicles on the bridge proper, it was estimated that the vehicular traffic for the first five years could be accommodated conveniently on a four-lane roadway. Thereafter, it probably will

be necessary to increase the capacity. While it is not expected that the bridge would eventually carry a traffic in excess of 20 000 000 vehicles, it was considered advisable to provide for ample margin by the doubling of the initial capacity, which would be sufficient to accommodate at least 25 000 000 vehicles annually. Moreover, the additional roadway lanes will make it possible to segregate slow and fast moving traffic and thus permit greater speed, safety, and convenience of travel.

While the predominant necessity of the bridge for vehicular traffic was recognized, the possibility that it might be of service to rail passenger traffic was given careful consideration. With present-day tendencies to transport people in automobles and buses, that mode can be relied upon to take care of whatever demand there will be for passenger transportation across this bridge for a number of years. There can be little doubt, however, that with the growing up of a large population contiguous to the bridge the more efficient and economical transportation of people by electric rail service will become a necessity. Whether, and how, this traffic is to be carried over the bridge is yet a subject for study by the proper transit authorities in the two States.

The studies of the Port Authority's Staff indicated that the prospective volume of traffic fully warranted the comparatively small expenditure which was necessary to provide for four rapid transit tracks on the bridge, and such provision, therefore, was made in the design. It is believed that this provision is ample and all that may be reasonably justified at the present time (1932).

Topographical Conditions, Surveys, and Borings

A superficial examination of the topography at the site selected indicates the favorable conditions for a bridge situated within the limits defined in the Legislative Acts. The ground on both sides is high; that on the New Jersey side reaches, at the top of the Palisades, only about 500 ft. from the shore line, an elevation of about 280 ft. (Fig. 9), and that on the New York side rises, on the Washington Heights ridge, to a general elevation of about 200 ft., approximately 1 000 ft. from shore (Fig. 10). This permits of comparatively short, inexpensive approaches (Fig. 11); and, furthermore, the location in that vicinity does not involve extensive destruction of highly improved property as compared with location farther south.

An elevation of the upper roadway on the bridge, of about 240 ft., leaving 200 ft. clear height under the bridge, therefore, conformed with the general topography and incidentally provided ample clearance for all shipping that is likely ever to pass under it.

A glance at Fig. 24, introduced subsequently, indicates that obviously, on account of the narrowing of the river, the location of the bridge at the extreme point of Fort Washington Park is the most favorable one, requiring the least length of span. However, in order to determine conclusively, within the limits defined by the State Acts, not only the most favorable exact location of the bridge, but also the most feasible and economical arrangement of the main structure and the approaches, and reliable estimates of cost, it was

essential to undertake an accurate preliminary topographical survey of the vicinity, including a triangulation across the river, and borings to establish the subsurface conditions, more particularly the depth to solid rock. The surveys were embodied in a large map to the scale of 1 in. = 100 ft., and subsequently used also for the preparation of detailed maps to the scale of 1 in. = 20 ft.

Sixteen borings, all carried well into the solid bed-rock, were made late in 1925 at three tentatively selected locations, namely, in the vicinity of 181st Street, 179th Street, and 175th Street, Manhattan, respectively. At all three locations solid rock was found near the westerly pier-head line at depths ranging from 115 to 170 ft., with the surface of the rock falling sharply toward the river. In the borings made 500 ft. and more beyond the westerly pier-head line and carried to a depth of 200 ft., only silt was encountered. On the New York side the solid rock bed forms the shore, but its surface also falls off sharply toward the river.

These borings confirmed the assumption that, between the pier-head lines then established by the War Department, the bed-rock is too deep to permit of economical construction of bridge piers and that such piers must, and can, be placed between the pier-head lines and the shore, or on the shore. Preliminary and comparative plans and estimates were based on the results of these borings, and they confirmed the superior economy of the finally selected location on a line between 178th and 179th Streets.

As soon as the plans for the financing were effectuated, late in 1926, thirty additional borings were undertaken at the selected site of the New Jersey Tower. These borings were recorded on a glass model and revealed a surface of rock with a fairly uniform general slope of about 30° falling toward the river, but with contours practically parallel to the shore, and with a depth, within the area of the pier foundation, ranging from about 35 ft. at the southwest corner to a maximum of 75 ft. in the northeast corner.

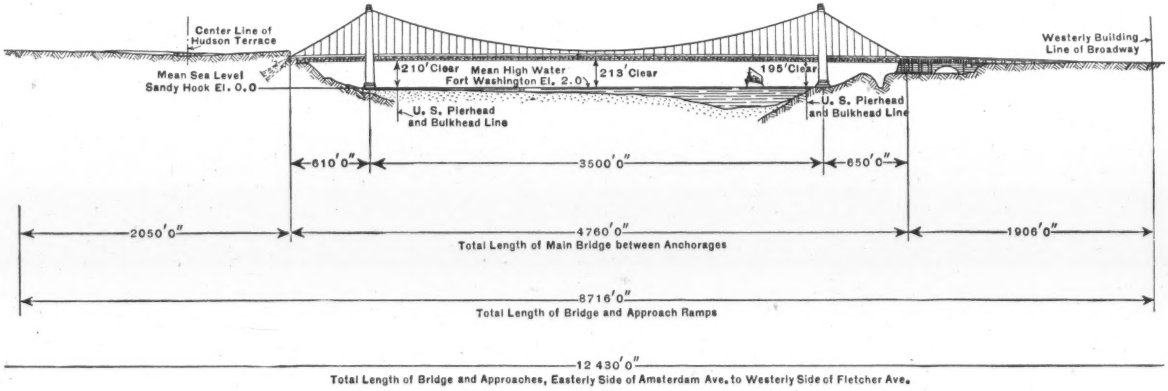
While the results of the final borings indicated the feasibility of shifting the assumed pier location slightly toward the river to a depth to rock of about 100 ft., and thus shortening the span, this was not considered advisable, nor of any material benefit. On the contrary, the fact that the maximum depth to rock was only 75 ft. and the average depth less than 50 ft., made it appear feasible to build the foundation within an open coffer-dam. This method of foundation was adopted upon recommendation of Daniel E. Moran, M. Am. Soc. C. E., Consulting Engineer on Foundations, in competition with the pneumatic process which, to that time, had been considered the most practicable and the safest method. As a result of competitive bidding, the coffer-dam method proved to be about $250 000 less expensive.

The borings also indicated, and subsequent exposure of the rock surface confirmed, a mostly hard and compact rock structure and an absence of open seams and large boulders which might have made the open coffer-dam method very difficult, if not impracticable.

Additional borings were made also at the sites of the tower and anchorage on the New York side, and they confirmed the assumption that both structures would rest on a solid bed of hard Hudson schist. An examination

FIG. 9.—"PALISADES CLIFFS," 300 FEET HIGH, ON SITE OF BRIDGE, ON THE NEW JERSEY SIDE.

FIG. 10.—"WASHINGTON HEIGHTS," MORE THAN 200 FEET HIGH, NEAR SITE OF BRIDGE ON MANHATTAN SIDE.


Center Line of Hudson Terrace
Mean Sea Level Sandy Hook El. 0.0
210' Clear
Mean High Water Fort Washington El. 2.0
213' Clear
195' Clear
Westerly Building Line of Broadway
U. S. Pierhead and Bulkhead Line
U. S. Pierhead and Bulkhead Line
610'0"
3500'0"
650'0"
2050'0"
4760'0"
Total Length of Main Bridge between Anchorages
1906'0"
8716'0"
Total Length of Bridge and Approach Ramps
12 430'0"
Total Length of Bridge and Approaches, Easterly Side of Amsterdam Ave. to Westerly Side of Fletcher Ave.

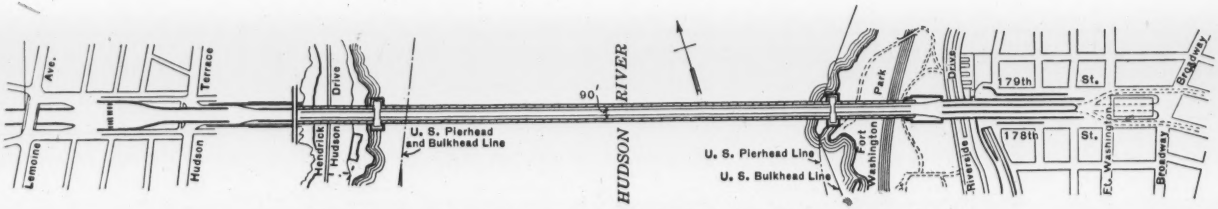


FIG. 11.—GENERAL PLAN AND ELEVATION, GEORGE WASHINGTON BRIDGE.

of the exposed rock forming the Palisades indicated, as was subsequently confirmed by the excavation, that that extremely hard and compact mass of trap-rock would offer an excellent anchorage for the bridge cables.

Geological Conditions

It was considered essential to establish, beyond doubt, that the various rock strata were sound geologically and of sufficient strength, solidity, and stability to sustain safely and durably the great loads to be imposed upon them or, in the case of the Palisades trap, to resist the enormous pull of the cables.

A very exhaustive study, based on the borings and examination of the rock exposed on the surface and, subsequently, in the excavation, was made by Charles P. Berkey, M. Am. Soc. C. E., who was retained as Consulting Geologist. Dr. Berkey constructed a diagram (Fig. 12) showing the relation

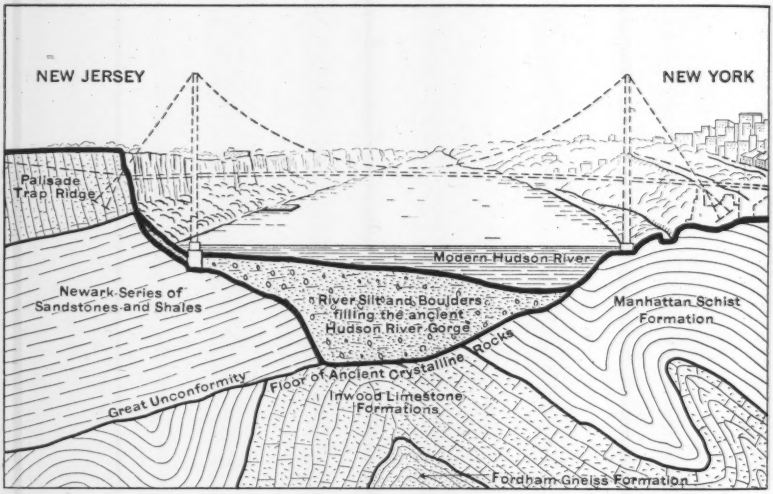


Fig. 12.—Geological Cross-Section at Site of George Washington Bridge.

of the various geological formations at the bridge site. This diagram which is distorted vertically, indicates a river gorge, filled with silt and boulders, perhaps more than 400 ft. deep, and reaching down to the previously unexplored ancient crystalline rock floor.

Overlying this floor on the New Jersey side, there is a bed, known as "Newark" formation (see Fig. 13), on which the New Jersey Tower rests. It is of sedimentary origin and is composed of a series of sandstones and shales of greatly varying strength and structure. (Compression tests of the cores showed a variation in strength of 3 000 to 24 000 lb. per sq. in. with a probable average of the entire mass underlying the tower base, of between 12 000 and 15 000 lb. per sq. in., or more than thirty times the maximum edge pressure of the tower base).

The borings indicated that the beds are slightly tilted, by an average of 10 to 15° from their original horizontal position, and that they dip toward the west beneath the Palisades. This is a very favorable condition in so far as the tower is concerned, in that it precludes any tendency of a portion of the bed-rock to slide toward the river gorge, such as might exist in case the beds would dip with a steep inclination toward the gorge.

This condition was confirmed by subsequent examination of the actual rock surface after it was exposed by excavation of the overlying loose material within the coffer-dam. The exposure of the rock surface also revealed (as was to be expected), a very jagged surface with sharp steps falling off toward the east. This condition undoubtedly has been caused by the erosion of the softer beds between the harder ones. It also has proved to be favorable for the tower foundation in that it obviated the cutting of artificial steps in the rock, which otherwise might have been necessary in order to avoid any tendency of the tower to slide on the rock surface toward the river. In fact, the surface of the solid rock, after removal of all loose and disintegrated portions, secured such an excellent bond with the concrete base, that it was deemed unnecessary to remove any part of the solid rock.

The exposed rock upon which the tower base rests, proved to be predominantly of the hard, coarse, sandstone type which has more than ample margin of strength to carry the load.

The Palisades trap, in which the New Jersey anchorage is embedded, overlies the Newark formation. As forecast by the geologist it proved—in the excavation for the 40-ft. approach cut and for the anchorage tunnels reaching down 250 ft.—to be a compact mass of hard diabase, an igneous rock of volcanic origin (Fig. 14). Only near the surface, as a result of long exposure, since the Glacial Age, is it found to be seamy and disintegrated.

All the ledges exposed on the New York shore are of the type known as Manhattan schist, a crystalline micaceous rock forming most of Manhattan Island. At the site of both the tower (Fig. 15), and the anchorage (Fig. 16), this rock was found by borings and subsequent excavation to be eminently sound and free of zones of local weakness which have given considerable trouble elsewhere.

Type of Bridge and Span Arrangement of Main Structure

Topographical and geological conditions indicating so clearly the points of support, it was a comparatively simple task to determine the type and general span arrangement of the main structure.

It was natural, in order to reduce the river span to an economical minimum, to place a pier on the extreme rocky point of Fort Washington Park which is also close to the United States pier-head line and from which the bare rock surface falls off steeply. For reasons already mentioned, a pier on the New Jersey side had to be located about 200 ft. back of the pier-head line, thus fixing the river span at 3 500 ft.

This great span has given rise to criticism that it was extravagant, and that a more economical structure was feasible and permissible by moving the

New Jersey pier about 800 ft. out into the river, which is fairly shallow for a considerable distance from the shore, although the rock surface falls off to a depth of more than 200 ft. at that point.

Without regard to the possible adverse effect on shipping by a pier in the river, and without allowance for the hazards and uncertain cost and time of construction involved in building a suitable pier foundation to such great depth, a conservative analysis of cost should convince any one that the adopted span arrangement is the most economical under the given conditions.

In an endeavor to determine the effect which the shifting of the New Jersey pier would have on the cost of the entire structure, the writer discovered that for the slope of the rock surface as found, the saving which could be effected in the superstructure by decreasing the central span was practically offset by the increased cost of the deeper foundation. This comparison would not be so favorable, however, to the greater span if the conception of a rigid stiffening system formed the basis of proportioning, because under that theory the cost of the superstructure increases at a much faster rate.

There appears to be a widespread, but unwarranted, impression in the minds of engineers and others that length of span is the predominant factor in the economy of a large bridge; whereas, in many cases, such as that of the George Washington Bridge, a careful and rational analysis of conditions and costs would indicate that length of central span is a lesser factor.

It was also quite obvious that the face of the solid Palisades cliffs, about 650 ft. westerly of the New Jersey pier, fixed the location of an abutment or anchorage of a suspension bridge. The high rocky ground in Fort Washington Park, on the New York side, offered a symmetrically located, but not quite as favorable, point for an anchorage on that side. This location of a hugh abutment brought forward well-intentioned misgivings on the part of those interested in the preservation of public parks, and a demand was made that the abutment be moved to the still higher ground easterly of Riverside Drive.

The demonstration that the use of Fort Washington Park would not be materially impaired, that a more easterly location of the anchorage would result in a much longer side span, with consequent dissymmetry of the main structure, and deep, conspicuous, stiffening trusses over the Park and Riverside Drive (which would be much more objectionable from the æsthetic point of view than a well-designed massive anchorage and arch structure over the Drive, and would add millions to the cost), finally appeased the opponents and secured the approval of the City Government to the proposed arrangement.

The great central span and the possibility of the construction of solid, comparatively inexpensive, cable anchorages should force any student of the economics of long-span bridges to the conclusion that a rationally designed suspension bridge would be economically superior to any other conceivable type, not considering its superior æsthetic merits in that particular landscape.

This might not have been so always, and it is quite evident that even to-day engineers have different conceptions as to the relative economic merits of different types for long spans. Indeed, if the many designs are compared,

FIG. 13.—NEWARK FORMATIONS OF SANDSTONE AND SHALES AT BOTTOM OF NEW JERSEY TOWER.

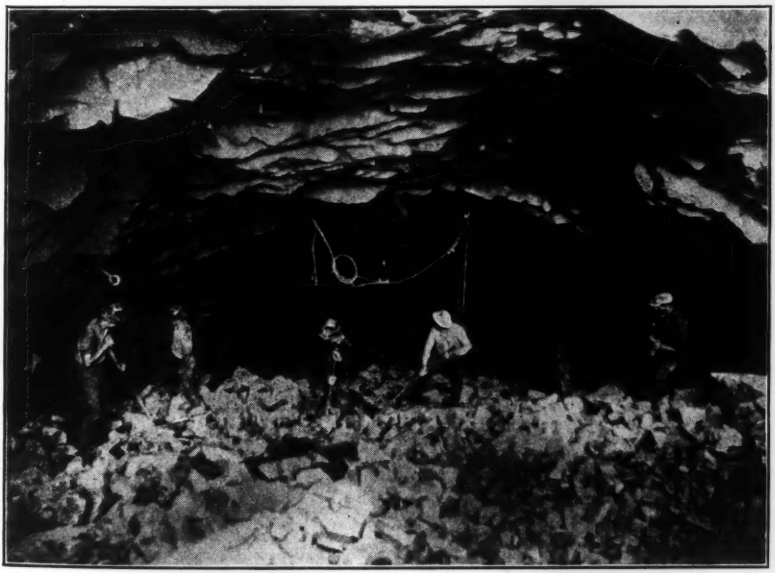


FIG. 14.—SOLID ROCK, OR TRAP-ROCK FORMATION, AT NEW JERSEY ANCHORAGE.

FIG. 15.—MANHATTAN SCHIST AT BOTTOM OF NEW YORK TOWER, NEAR SURFACE.

FIG. 16.—BED OF MANHATTAN SCHIST AT SITE OF NEW YORK ANCHORAGE.

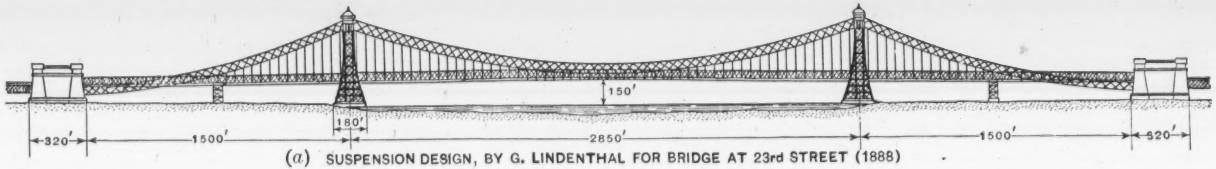
which from time to time have been proposed for a bridge across the Hudson at New York, or across other wide navigable streams, a perplexing diversity of conceptions is found of the best design for such a long span. (See Figs. 17, 18, and 19.)

The cantilever, pure or hybrid, may be dismissed as a possibility by referring to the exhaustive investigations of the War Department in 1894, as referred to elsewhere in this paper. The structure then proposed by the New York and New Jersey Interstate Bridge Company was to be a cantilever (Fig. 17(*c*)) with a central span of 2 000 ft. Comparing this with a suspension bridge of a type shown in Fig. 18(*a*), the Board of Engineers found that even for a central span of 3 200 ft. the suspension type would not be materially more expensive than the proposed cantilever.

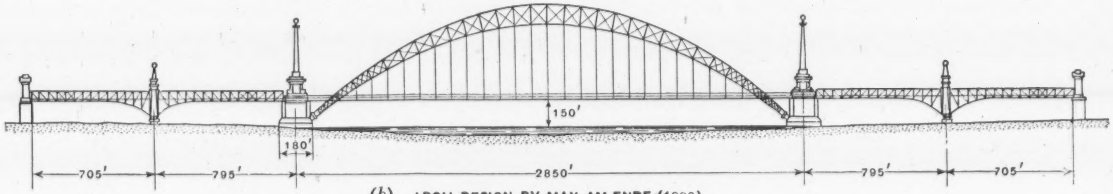
Under present-day conceptions of the rational design of the two types of bridges, and in particular under conditions such as exist at the George Washington Bridge, the economic difference between the two types is materially accentuated in favor of the suspension type. The superiority of the suspension type over the arch, both economically and æsthetically, is less obvious, and the site of the George Washington Bridge, with its solid rocky shores, might invite an investigation of the latter type. The comparison of an arch with a span of 1 685 ft. over the Kill van Kull in 1928 proved this type to be more economical than that which was considered an equivalent suspension bridge with a central span of 1 522 ft., largely because of expensive anchorages required by the latter on account of the low level of the ground. At Fort Lee, both arch abutments would have to be set several hundred feet back of the location of the suspension bridge towers—on the New York side to satisfy clearance requirements for shipping, on the New Jersey side to find solid rock nearer the surface—and the span of an arch would thus become close to 4 000 ft. This, with the relatively inexpensive cable anchorages on account of the high rocky ground and the increasing economy of a suspended structure with increasing span, should leave no doubt of the economic superiority of the suspension type at Fort Lee.

A remarkably bold and very creditable design for an arch bridge across the Hudson River was made in 1889 by the English engineer, Max am Ende, (Fig. 17(*b*)). He claimed at the time that his arch would be more economical than the suspension type proposed by Mr. Lindenthal. In accordance with present-day conceptions in design there can be little doubt that such an arch would be very much more expensive, and a great mass of steel at the height of about 600 ft. above water level would not be as attractive in appearance as a graceful suspension bridge, provided the latter is without clumsy stiffening trusses, such as were embodied in several of the early designs for a Hudson River bridge.

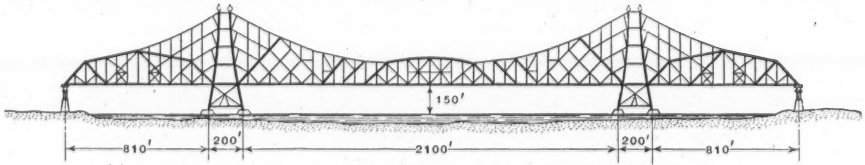
All later designs show preference for the suspension type. For a span of 3 500 ft., and under conditions permitting a relatively light stiffening system and inexpensive anchorages, as in the case of the George Washington Bridge at Fort Lee, the suspension type is unquestionably far more economical than any other type.



(*a*) SUSPENSION DESIGN, BY G. LINDENTHAL FOR BRIDGE AT 23rd STREET (1888)

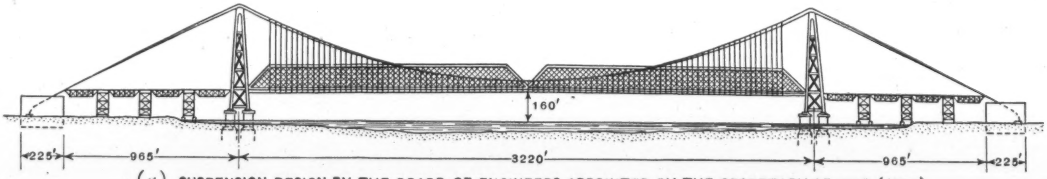


(*b*) ARCH DESIGN, BY MAX AM ENDE (1889)

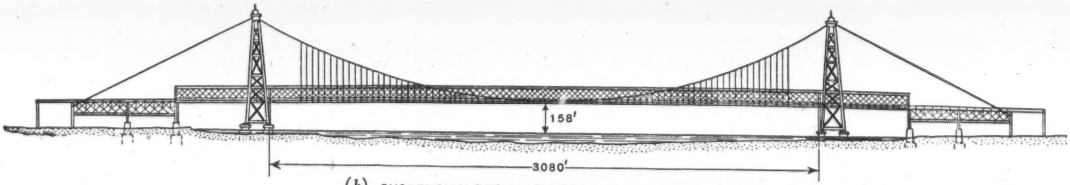


(*c*) CANTILEVER DESIGN, BY UNION BRIDGE CO. FOR BRIDGE AT 70th STREET (1893)

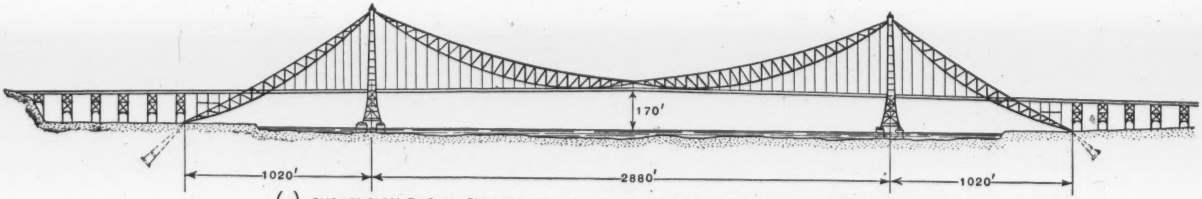
FIG. 17.—EARLY STUDIES FOR A BRIDGE ACROSS THE HUDSON RIVER.



(*a*) SUSPENSION DESIGN BY THE BOARD OF ENGINEERS APPOINTED BY THE SECRETARY OF WAR (1894)



(*b*) SUSPENSION DESIGN BY GEORGE S. MORISON (1896)



(*c*) SUSPENSION DESIGN BY BOLLER, HODGE AND BAIRD FOR BRIDGE AT 57th STREET (1913)

FIG. 18.—EARLY STUDIES FOR A BRIDGE ACROSS THE HUDSON RIVER.

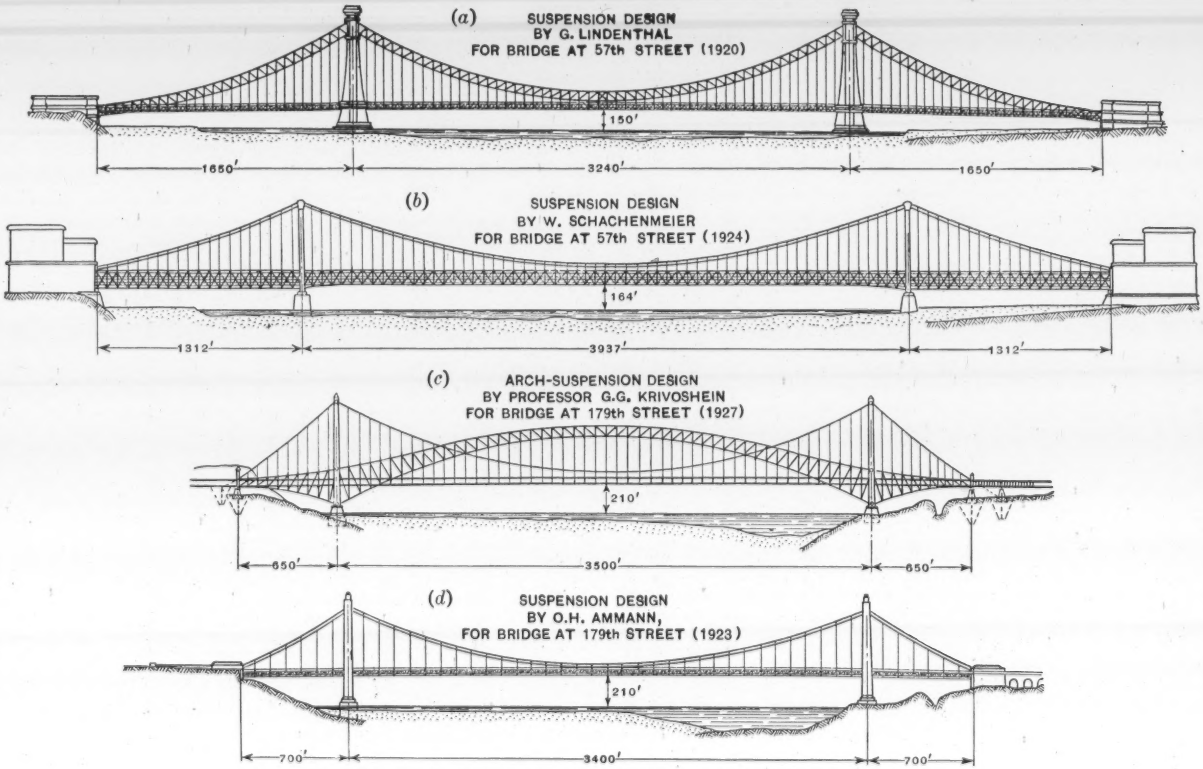


FIG. 19.—RECENT STUDIES FOR A BRIDGE ACROSS THE HUDSON RIVER.

From time to time there have been proposed and likewise found application combinations of the pure suspension type and other types. Whatever may be the claims of scientific or economic merits of such hybrid types (and it is very doubtful in the writer's mind that they are justified because of the lack of structural simplicity and clearness of function of such incongruent systems), they cannot satisfy æsthetic principles.

Fortunately, while such types have found their way into the field of moderate spans where they form interesting structural creations, they have not advanced beyond proposals, in the field of long spans. Even Mr. John A. Roebling, troubled by the problem of finding means to provide sufficient rigidity, seriously considered a combination of a rigid truss with suspended cables, but his good common sense finally led him to the simple suspension design so admirably illustrated by the Brooklyn Bridge.

Fig. 19(*c*) shows an interesting design of a combination arch and suspension type, proposed in 1927 by Professor Gregory G. Krivoschein, of Prague, Czecho-Slovakia, to fit the location of the George Washington Bridge with a span of 3 500 ft. A combination of rigid cantilever truss with suspended cables and span of 3 900 ft. is illustrated in a study made in 1924 by the late Professor W. Schachenmeier, of Munich, Germany, to fit the location of a bridge across the Hudson River near 57th Street, Manhattan. (Fig. 19(*b*)).

Maximum Feasible Spans.—The unprecedented length of span of 3 500 ft. being exactly twice the longest suspension span in existence has given rise to the question on the part of laymen as to whether the building of this bridge is feasible.

Engineers familiar with the design and construction of large bridges have pointed out from time to time that the feasibility of building a bridge of a span as long as 3 500 ft. and more, is essentially a question of economy, and that the span length and size of a bridge have nothing whatever to do with its safety, either during erection or after completion.

The feasible limit of span is reached when the metal required to carry a given load becomes excessive in cost and not because the safety is impaired. The physical limit of span is reached when no amount of metal can safely carry more than its own weight. The latter limit can be mathematically determined for the safe strength of any given material, and load conditions. In the aforementioned investigation in 1894 by the Board of Engineer Officers, 4 335 ft. was found, upon conservative assumptions, to be "the maximum span practicable from the engineering point of view."

In acordance with present-day accepted views regarding the proportioning of stiffening trusses in long spans, the formula used by that Board results in greatly excessive weights of these trusses, and of the cables and towers which have to carry them. Furthermore, the permissible stress in the wire cables was assumed at only 60 000 lb. per sq. in., whereas with material of 240 000-lb. strength available to-day, and for very long spans, a permissible stress of 90 000 or 100 000 lb. per sq. in. would be entirely safe. In the light of these facts it may be demonstrated easily that a modern bridge of 10 000 ft. span could be built with perfect safety. It is only above this limit that the

FIG. 19.—RECENT STUDIES FOR A BRIDGE ACROSS THE HUDSON RIVER.

cables increase rapidly in weight and cease to be practicable, if they are to carry any appreciable load in addition to their own weight. The practicable length of span for a cable to carry itself is, of course, much greater.

Conception of Type of Suspension System.—Thus, while the type of bridge and its span arrangement logically forced themselves upon the designer, he was yet confronted with the more complex and controversial questions of selecting the appropriate form and proportions of the suspension system. The great number of different systems which have been proposed and applied, indicate a wide diversity of conceptions and practices in respect to these questions. Figs. 17, 18, and 19 show only a few systems, those variously proposed for a bridge across the Hudson River, at New York City.

In the case of the George Washington Bridge, the controlling criteria in selecting the system and its proportions were structural simplicity, maximum economy consistent with the required degree of rigidity, competitive conditions, and æsthetic conception.

The first of these criteria led to the system which is unquestionably the simplest in its structural details, as well as for erection, namely, the plain cable with parallel chord stiffening trusses along the floor; but this system, whether the trusses be three-hinged as in the design of the Board of Engineers (Fig. 18(*a*)), or two-hinged as in most of the modern suspension bridges, or continuous through the towers, as in the design of Mr. George S. Morison (Fig. 18(*b*)), or cantilevers, as in the study of Professor W. Schachenmeier (Fig. 19(*b*)), is not economical in a long span, nor in conformity with the writer's conception of a graceful structure, if, as in some of the designs, the stiffening trusses are made very deep.

Furthermore, it is not necessary, in accordance with more recent views, to provide deep and rigid trusses in a long heavy span, and, particularly so, when the side spans are relatively short and the dead load relatively great, as in the case of the George Washington Bridge.

Extensive studies of the relative rigidity of similar structures and their behavior under actual conditions, and calculations of the degree of rigidity to be obtained in the selected system, led the writer to determine finally upon a very shallow and flexible truss, which not only resulted in far-reaching economy, but also effected a light and graceful appearance Fig. 19(*d*)). Incidentally, by keeping the top chords of the stiffening trusses in the plane of the upper or roadway floor, obstruction by these trusses of the splendid view of the landscape that will be had from the roadway was avoided.

Where a considerable degree of rigidity is required, the trussed system, as proposed by Mr. Lindenthal (Figs. 17(*a*) and 19(*a*)) and (to a lesser degree) the stiffened chain system as proposed by Messrs. Boller, Hodge, and Baird (Fig. 18 (*c*)), are economical as far as material is concerned, but the erection of such suspended trusses unquestionably involves difficult and expensive erection operations and partly offsets the economy in material. Furthermore, in a large bridge, built by a public agency, it is essential that the widest possible competition be assured for all important parts of the structure, and the fact that the design for the George Washington Bridge lent itself equally

well to the use of wire cables and eye-bar chains was, therefore, an important factor in its adoption, and proved to be of decided advantage.

Finally, whatever influence these various considerations may have had on the general conception of the design, the writer has admittedly been influenced by his personal conceptions and taste. He has always been an admirer of the early English suspension bridges with their general simple appearance, their flat catenary, light, graceful, suspended structure, and their plain massive and, therefore, monumental towers.

Deviations from the simple unstiffened cables were due to the efforts to give the system greater rigidity. This has been accomplished by various more or less efficient expedients, such as inclined stays, connecting the floor directly with the top of the towers or other fixed points, by stiffening trusses placed along the floor or by stiffening systems attached to the cables themselves. In some cases the cable has even been combined with an upright rigid arch or with a cantilever truss.

It is significant, however, that after nearly a century of efforts to devise and introduce novel forms of suspension systems, or hybrids between the suspension type and other types, engineers, in designing the longest modern suspension bridges, have returned or adhered to the simple, naturally graceful forms which are characteristic of the early bridges of this type.

The Floor Structure, Stiffening Trusses, and Wind-Bracing

Consideration of the traffic requirements, the conception of the stiffening system, and the arrangement of the four cables in pairs on each side of the floor, led to a structurally and statically simple arrangement of the floor system suspended from the cables (Fig. 20).

An upper floor is designed to accommodate at least eight lanes of vehicular traffic. Beneath it, and connected to it by rigid floor frames, is a lower deck designed to carry at least four tracks for heavy rapid transit traffic; or this deck may be utilized for additional vehicular lanes in case that should ever become necessary and desirable.

A shallow stiffening truss with chords only 29 ft. apart vertically is placed on each side of the floor system in the plane of the cables and suspenders. A single, relatively flexible, horizontal wind truss is arranged in the plane of the upper deck, the upper chords of the stiffening trusses forming the chords of this horizontal wind truss. The wind forces acting on the lower deck are transmitted to this wind truss through the rigid floor frames.

In a preliminary design the floor frame was conceived as an inverted **U**, with brackets cantilevering out from the vertical posts on both sides (Fig. 6). This arrangement was eventually abandoned in favor of the somewhat simpler and only slightly more expensive closed frame carrying all tracks inside the posts. This entire floor structure is designed so that the lower deck, together with the webs and bottom chords of the stiffening trusses, could be omitted initially and added in a very simple manner at any time in the future when necessity therefor will arise.

The vertical stiffening trusses have a depth of 29 ft. throughout, which is only one-one-hundred-twentieth of the central span. This compares with

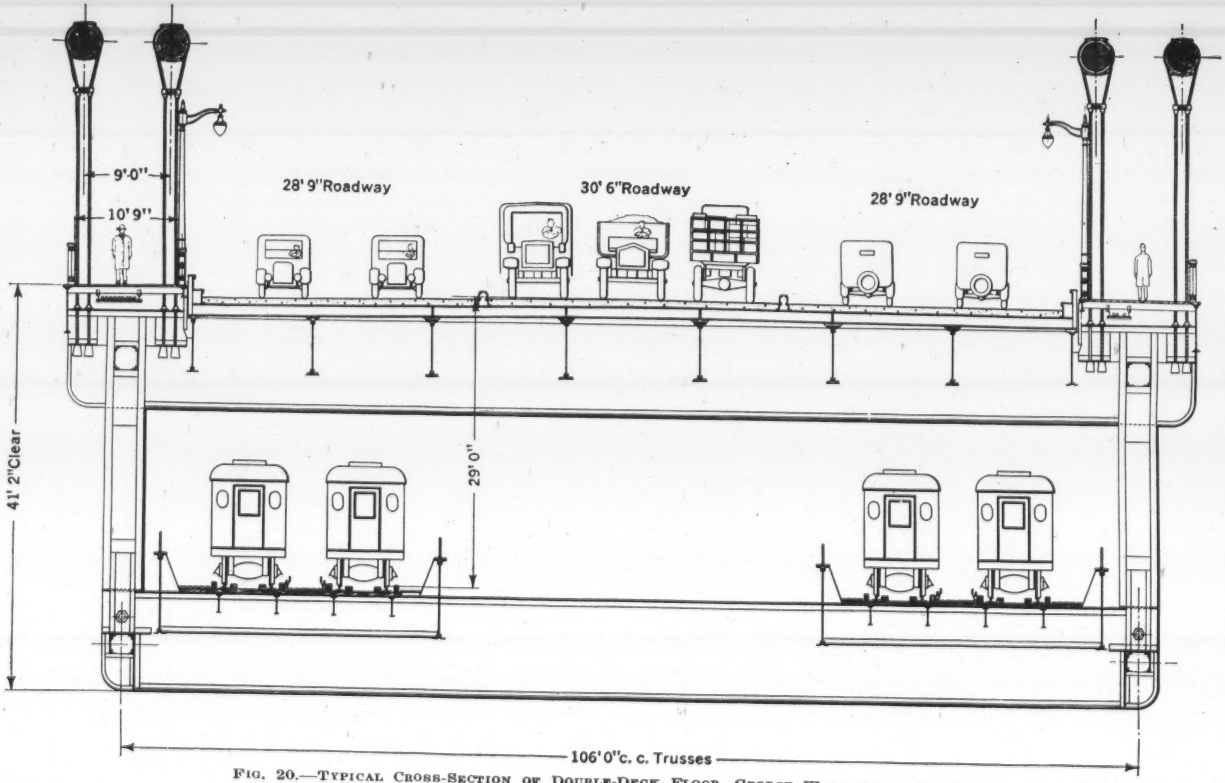


FIG. 20.—TYPICAL CROSS-SECTION OF DOUBLE-DECK FLOOR, GEORGE WASHINGTON BRIDGE.

the corresponding ratio of one-sixty-third in the Delaware River Bridge, which has a center span of 1 750 ft.; with one-sixtieth in the Manhattan Bridge, which has spans of 1 470 ft.; and with one-sixty-third of the central span of 1 630 ft. of the Bear Mountain Bridge, these being the three longest modern suspension spans in existence, or in course of construction, at the time the writer made the first studies. The first two, like the George Washington Bridge, are designed to carry highway and electric rail passenger traffic, while the Bear Mountain Bridge carries highway traffic only.

The Ambassador Bridge, in Detroit, Mich., built since that time, with a center span of 1 850 ft., has stiffening trusses 22 ft. deep, or one-eighty-fourth of the central span. The Golden Gate Bridge, in San Francisco, Calif., is designed with stiffening trusses 25 ft. deep, or one one-hundred-sixty-eighth of the central span of 4 200 ft. Both the latter bridges are designed for vehicular traffic only.

Stiffening trusses of the Manhattan and Delaware River Bridges may be called semi-flexible, while those of the George Washington Bridge are practically "flexible," that is, they exert almost no restraint upon the distortions of the unstiffened cables. On the contrary, they are forced to deform almost to the full extent and shape of the distortions of the unstiffened cables. The system in its initial stage, being without lower floor and stiffening trusses, is entirely flexible, that is, without any stiffening effect upon the cables.

The permissibility of an almost flexible system in the case of the completed bridge—that is, with rapid transit trains running over the bridge on the lower deck, or of an entirely flexible system in case the bridge carries only vehicular traffic on the upper deck—was not obvious to the writer at the inception of his studies.

The general tendency, both in the United States and abroad, had been toward a rigid stiffening system, and the textbooks and many modern treatises on suspension bridges had confined themselves entirely to that system and to the elastic theory, without respect to span length, dead weight of bridge, or character of traffic.

Extensive studies convinced the writer that for a long-span suspension bridge a rigid system was not necessary. He was also familiar with the fact that by the application of the correct or so-called deflection theory, as distinguished from the "elastic theory," to a more or less flexible system, material economies can be effected. This is inherently due to the stiffening effect of the dead load, which effect is ignored in the so-called elastic theory. The latter fact had been pointed out by various writers, notably, Professor Melan, of Vienna, Austria. It had also been proved by application of a modified deflection theory by Leon S. Moisseiff, M. Am. Soc. C. E., to the design of the Manhattan and Delaware River Suspension Bridges.[10]

Aware of the ample rigidity of the Manhattan Bridge under actual traffic conditions, and of the sufficiency even of the much more flexible Brooklyn Bridge under all ordinary conditions, the writer became convinced that in the George Washington Bridge—with its much longer and heavier center

[10] Final Rept. of the Board of Engrs. on the Delaware River Bridge, 1927.

FIG. 20.—TYPICAL CROSS-SECTION OF DOUBLE-DECK FLOOR, GEORGE WASHINGTON BRIDGE.

span (the latter weighing four times that of the Manhattan Bridge and ten times that of the Brooklyn Bridge), its comparatively shorter side spans with almost straight cables acting as rigid back-stays, and the more effective distribution of concentrated loads by reason of its wide rigid floor—stiffening trusses of relatively greater flexibility than those used in the aforementioned smaller bridges, were permissible and economically required.

As a result of lengthy theoretical investigations, supplemented by observations on mechanical models, made in an endeavor to find the appropriate degree of rigidity of the stiffening trusses for the George Washington Bridge, the writer came to the conclusion that the arrangement of nearly flexible trusses in the finished bridge, and the omission of trusses in the initial stage of a single highway deck, were perfectly permissible and would secure a degree of rigidity at least equivalent to that of any of the aforementioned large modern bridges.

In fact, the governing function of a stiffening system in a long span is to prevent excessive gradients of the floor due to deflection of the cable. For vehicular and electric passenger traffic the limiting grade, under severe loading conditions, can safely be assumed at 5%, in view of the improbability that it would ever be produced, and even then it would prevail only over very short stretches.

The deflection curves for the George Washington Bridge showed that, under a combination of extreme temperature change and live load, the grades in the perfectly articulated bridge would not exceed 2.4%, and in the stiffened bridge as designed would be less than $2\frac{1}{4}$ per cent. The function of the comparatively light flexible stiffening trusses was considered mainly to stiffen the floor locally.

In his studies the writer became aware of the fact that even the so-called exact or deflection theories (a number of which had been advanced by that time), became unreliable and inapplicable in the case of trusses of relatively great flexibility, and that the simplest and most reliable method was to calculate the deflections of the unstiffened cables and the corresponding bending moments produced in the trusses, and, subsequently, correct the results slightly to allow for the comparatively small stiffening effect of the trusses.

The economy effected by adopting flexible trusses is far-reaching, as may be seen by a comparison of the weight of stiffening trusses and wind-bracing compared to the weight of cables, the live load, and the total dead load per foot of bridge in the George Washington Bridge and three other bridges (see Table 2).

In the Manhattan Bridge the stiffening system amounts to 30% of the entire dead load and nearly 140% of the main carrying members, the cables and suspenders. In the completed George Washington Bridge the stiffening system amounts to less than 6% of the entire dead load and less than 20% of the weight of the cables and suspenders.

Some formulas would increase the weight of stiffening roughly in proportion to span and live load. On that basis, when compared with the stiffening of any of the bridges in Table 2, the system of the George Washington Bridge, with silicon steel chords, would weigh roughly from 13 000

to 14 000 lb. per ft. Actually, it is about one-sixth this comparative weight in the final condition and about one-twelfth in the initial condition. Considering also the fact that every dollar spent for steel in the floor and stiffening trusses in a span of this length requires at least an equivalent expenditure

TABLE 2.—COMPARISON OF DEAD LOAD WEIGHTS

Item	Manhattan Bridge	Bear Mountain Bridge	Delaware River Bridge	George Washington Bridge
Span length, in feet	1 470	1 630	1 750	3 500
Approximately equivalent live load, in pounds	11 000	2 500	7 000	8 000
Average weight of stiffening trusses and wind bracing, in pounds:				
For highway only	.....	2 650	.....	1 110
For highway and rapid transit	7 200	.....	5 730	2 350
Grade of steel	Nickel, in chords	Silicon	Nickel, in chords	Silicon, in chords
Average weight of cables and suspenders, in pounds	5 300	1 500	4 650	12 530
Total average dead load, in pounds	24 000	11 500	23 700	40 000

for materials in the cables, towers, and anchorages to carry the floor steel, the total saving by the adoption of the flexible trusses is estimated to be almost $10 000 000.

This saving in the stiffening trusses of the George Washington Bridge is not due entirely to their flexibility in a vertical plane, but somewhat to the comparative flexibility of the horizontal wind trusses the chords of which form the upper chords of the vertical stiffening trusses. Due to this lateral flexibility, and to the great weight of the cables, a large proportion of the wind load is transmitted from the wind truss to the cables and by the latter to the tops of the towers. Thus, the wind trusses become much lighter, and this economy, together with the corresponding saving of materials in the cables, towers, and anchorages, is only slightly offset by the extra material required in the towers on account of the increased lateral cable reaction at the top.

The studies indicated that under possible severest wind pressure acting over the entire length of the center span of 3 500 ft. (heaviest wind pressures have been observed to extend generally over a width not greater than 800 ft.), the maximum lateral deflection at the center, if the bridge were perfectly articulated and without lateral bracing, would not exceed 12 ft., or about one-three-hundredth of the span length, which would be entirely permissible. Actually, the rigidity of the floor and the inertia of the enormous dead weight resisting sudden gusts of wind prevent even such deflections, and the bridge as a whole would be perfectly safe and sufficiently rigid even without a wind truss. The comparatively flexible wind truss was provided mainly to prevent possible excessive local distortion of the floor and consequent high stresses in the floor members and their connections as a result of extraordinary wind effects.

Although no measurements of deflections have been made to date (1932), observations even on the partly completed bridge indicate plainly its remarkable lateral rigidity.

A fuller description of the stiffening system is contained in another paper of this series. Reference is made also to a more complete discussion by the writer of the nature of the stiffening system of suspension bridges and its effect upon the economy of such bridges in his discussion[11] of the paper by J. A. L. Waddell, M. Am. Soc. C. E., entitled "Quantities of Materials and Costs per Square Foot of Floor for Highway and Electric Railway Long-Span Suspension Bridges."

The Cables and Their Anchorages

The conception of the type and arrangement of the cables is intimately tied up with that of the floor structure. As mentioned, it was desired to suspend the latter only in two vertical planes to secure greatest structural simplicity and determinateness of stress action. Owing to the consequent large concentration of load in each of the two planes of suspension, it was evident that, instead of a single large cable of a size far beyond any previously constructed, it was advisable to arrange a group of two or more cables. Such division into units was considered called for, also, because it was held possible that part of the group of cables would be erected first, and the remainder later, when increase in traffic capacity became necessary and justified the additional expenditure. This idea was finally abandoned in favor of the initial completion of the cables for full load capacity, although it involved an additional initial expenditure of about $8 000 000.

Such an arrangement raises the question of equal distribution of load between the various units of the group, whether they are separate wire cables or units of eye-bar chains. Not only laymen, but engineers have been puzzled over this question, and complicated structural devices have been proposed, and actually used, in order to secure uniformity of distribution of load and stress; yet this is one of the simplest problems and one which solves itself naturally where the cables or their units are free to deflect, because if the suspension of the floor is arranged so that the cable units must deflect equally, their stresses from vertical load must be the same. The uniformity of distribution is the greater the longer the span and the flatter the catenary.

Inequality of stress is caused mainly by inequality of temperature changes in different parts or units of a cable. Such temperature variations cannot be avoided, either in a single cable or a group of cables, and must be taken care of in the marginal strength; but it is the writer's belief that their effect has been largely exaggerated. In the case of the George Washington Bridge, various schemes of grouping the cable units and their sequence of construction were investigated and in part left for selection by the contractor. The arrangement finally determined upon provides for a pair of cables, 36 in. in diameter, on each side of the floor, the individual cables lying side by side, 9 ft. apart on centers. The type of suspension system selected lent itself to two radically different types of cables, the so-called wire cable and the so-called eye-bar cable or chain. In fact, this adaptability of the general design of the bridge to both these types was an important factor in its conception.

[11] *Transactions*, Am. Soc. C. E., Vol. 91 (1927), p. 932.

In a structure of this magnitude, especially when undertaken by a public body, and under conditions which prevailed in this particular case, the widest possible competition is essential and it was for this reason that upon recommendation of the writer, concurred in by the Consulting Engineers and the General Manager, the Commissioners of the Port Authority approved the preparation of competitive designs and the calling for competitive bids, not only for the two different types of cables, but for various alternate arrangements, such as cables of each pair placed side by side or one above the other; also, for cables erected simultaneously or successively, thus giving the bidders wide latitude in applying their ingenuity, experience, and facilities in developing the most economical methods of fabrication and construction.

The outcome of the bidding, which was in favor of the wire cable type as the lowest in cost to the Port Authority, indicated clearly that this course fully justified the additional time and expenditure involved in the preparation of competitive designs and was to the best interest of the Port Authority and that of the public. Such procedure, however, might not always and in all cases be justified, and the relative prices established by this competition may be taken as a fair basis for comparative studies for other projects at the present time (1932).

The question of the relative merits of the two types of cable has been the subject of considerable discussion and controversy among engineers from time to time, in the United States and abroad. It is a complex question and one which lends itself legitimately to personal preference, whether that be based upon impartial judgment, prejudice, or interest.

In so far as engineers are concerned, their impartial judgment (considering present knowledge based upon past performance of the two types), must take into consideration such important factors as the peculiarities of the case, the relative quality of material permissible, unit stresses, and other details of design. General condemnation of one or the other type on the vague grounds of greater or lesser weight, inferior safety, durability, or assumed greater cost, is open to justified criticism of partiality or prejudice.

In so far as the public is concerned, there are the outstanding facts that both types belong to the oldest forms of bridge members. Both have been developed in American bridge practice during more than a century; both have reached a high degree of perfection in material and structural details; and both have been used in the largest American bridges built in the twenty years since about 1910. There are bridges in existence, built more than 100 years ago, of both types, and there is no evidence to show that when properly proportioned and adequately maintained, either type, as a type, is not safe and will not endure for an indefinite time. Both types have been advocated and proposed for long-span bridges by engineers of prominent standing, and both types are being manufactured by American manufacturers of the highest reputation and long experience.

Finally, those who would unqualifiedly condemn the eye-bar cable should consider the fact that any wire cable is in reality a combination of wire cable and eye-bar chain, because for structural reasons all the important cables

thus far built terminate in eye-bar chains which differ in no essential element from those which would constitute the main cables.

The cost depends very largely upon circumstances. Early studies for the George Washington Bridge, based upon equitable unit stresses and design details, and upon prices which were then established and indicated by information from the manufacturers, showed plainly that competition between the two types might result in a considerable saving in favor of the eye-bar; and, indeed, if one examines the bids and considers the unit prices which prevailed prior to the competitive bidding one is forced to conclude that this assumption was justified.

A feature which in this particular case favored the economy of the eye-bar cable is the short, steep, back-stays which resulted from the peculiar arrangement of the spans. Such a condition involves a considerable excess section in the wire cable by reason of the practical necessity of making the cable section uniform throughout, while the section of the eye-bar cable can be varied with the stress.

It was suggested that the back-stays be made longer and that that they be given an inclination equivalent to that of the center span cable at the tower. This would have resulted in a materially greater cost for either type and would have made a monstrous looking structure.

The general design and arrangement of the cable anchorages resulted largely from the given topography and geological formations. On the New Jersey side, the hard compact basalt or "trap rock" formation of the Palisades offered, as the logical and most economical solution, inclined anchorage tunnels driven into the rock formation below the floor and refilled with concrete after the placing of the steel anchorage structures to which the cables are attached. On the New York side, the point of intersection of the cables and floor is approximately 100 ft. above the rock surface in Fort Washington Park. A continuation of the cables below the floor and their anchorage in the rock structure was considered, but was found to be neither economical, nor æsthetically desirable, and the anchorage of the cables in a masonry block built up from the ground to the bridge floor was determined upon as the best solution.

This block, although in itself a huge and massive looking structure, forms also a natural abutment for the great arch over Riverside Drive and blends well with the surrounding landscape. Its appearance will be enhanced by appropriate architectural treatment of its surfaces.

The Towers

There is no part of the design of the George Washington Bridge which has called forth as much comment, favorable and unfavorable, on the part of engineers, architects, and laymen, as the towers. Indeed, as the writer has endeavored to show, the design of the suspended structure, the floor, and the cables, resolved itself largely in the application of natural and most simple structural forms which neither required nor permitted architectural treatment to satisfy æsthetics.

The design of the towers, however, is not so well defined. There are widely different meritorious forms and the effect of the towers upon the appearance of the entire structure is perhaps more pronounced than that of any other part. They may enhance or destroy the natural beauty of a graceful suspended structure. There are existing examples which illustrate both effects.

It is futile to theorize about this question—it is largely a matter of æsthetic conception, which is so intensely individual and changeable—nor can it be dealt with on general principles without regard to the local scenery or landscape. Moreover, the æsthetic treatment of a bridge, as that of any other engineering structure, is not always satisfactorily solved even by correct and honest application of engineering principles. The appearance of a structure so conceived may sometimes be materially enhanced by the addition or the architectural embellishment of certain structural parts, whether structurally required or not. The flanking abuments of an arch bridge, and the towers and the anchorages of a suspension bridge, offer opportunity for such enhancement.

At the time the design for the George Washington Bridge was conceived a general tendency prevailed to design the towers of large suspension bridges as steel frames or bents, slender as seen in the elevation of the bridge, with fixed base, but with sufficient flexibility to stand bending resulting from the longitudinal motion of the tower tops to which the cables are fixed.

This type, first successfully applied and architecturally well conceived in the case of the Manhattan Bridge, undoubtedly answers engineering requirements best where great flexibility is required in the case of long side spans, and in many localities (if well designed) it fully satisfies æsthetic requirements. Unfortunately, in a number of cases, such towers have been so crudely designed as to be responsible for much of the adverse criticism of steel towers in general on the part of the art-loving public.

There are existing towers of this kind which are manifestly too slender. Flexibility is not a virtue where it is not needed. Excessive flexibility may cause high secondary stresses which are apt to be ignored, and extremely slender towers give an impression of weakness, but no matter how well designed such slender steel towers may be, and how much they may be justified in certain cases, they can not compare in their monumental effect upon the entire structure with the massive towers so admirably exemplified in the Brooklyn Bridge and in many of the older suspension bridges.

The Brooklyn Bridge, long since its completion surpassed in size and technical achievement, still retains a world-wide reputation as the most fascinating and outstanding structure of its kind, and there can be little doubt that this is due to its admirable gracefulness, coupled with the monumentally conceived towers.

Is it surprising then that in spite of the present tendency toward purely utilitarian and so-called scientifically correct structural forms, repeated efforts should be made on the part of engineers and architects (and such efforts will continue to be made), to produce the effect of massive towers by designs adapted to the use of present-day available materials of construction.

In the case of the George Washington Bridge—owing to its location in a landscape which it is hoped will forever retain its natural beauty, with a background of massive cliffs on one side and the rocky and wooded promontory forming Fort Washington Park on the other—massive looking towers for the bridge appeared particularly well adapted.

As to the adaptability and economy of materials available to produce massive towers of such great height, bridge builders now have, in the combination of steel and concrete, a means of producing much greater strength at much less cost than would be possible with the formerly available stone masonry. This form of construction has made such successful inroads in the field of long-span arches, that its economical application to towers of great height appears logical and feasible.

In fact, comparative studies of steel towers and massive towers made for the George Washington Bridge indicated that at prevailing prices a massive tower of steel and concrete can be designed that will compare very favorably with bare steel towers. A study made by the writer for a slender steel tower (Fig. 21) was given serious consideration in comparison with a massive tower, but, while it might have been a fairly satisfactory sclution, it was the unanimous opinion of those responsible for the design, that in this case it was less meritorious from the æsthetic point of view than the design for the massive tower eventually adopted.

In line with the plan to construct the bridge substantially in two stages of traffic capacity, and to effect the maximum economy, the composite tower was designed as a self-contained steel frame or skeleton which would support itself and, at least, the initial dead load of the suspended structure, but would be encased and strengthened by concrete properly bonded to the steel skeleton and reinforced with reinforcement steel, to obtain the full ultimate carrying capacity of the bridge.

Owing to the great height and comparative slenderness of the massive towers as designed, and the relatively short side spans which govern the longitudinal motion of the cables at their bearings over the towers, it was found feasible, without subjecting the tower to undue stresses from bending, to fix the cable bearings to the towers. Thus, it was possible to avoid movable bearings which, if designed so as to be permanently effective, would be very expensive.

In the course of the final studies for the towers, differences of opinion arose as to the ultimate strength of the combined steel and concrete structure. In order to avoid any controversy about this question and to permit the postponement of the final decision regarding the design without delay to the construction of the steel skeleton, it was decided finally to proportion the steel skeleton so that it would be capable of carrying the entire ultimate load without reliance upon the concrete encasement or else to permit the construction of an independent concrete shell around the steel skeleton if ultimate studies should prove such a solution to be justified. This strengthening of the steel skeleton involved an additional initial expenditure of approximately $800 000.

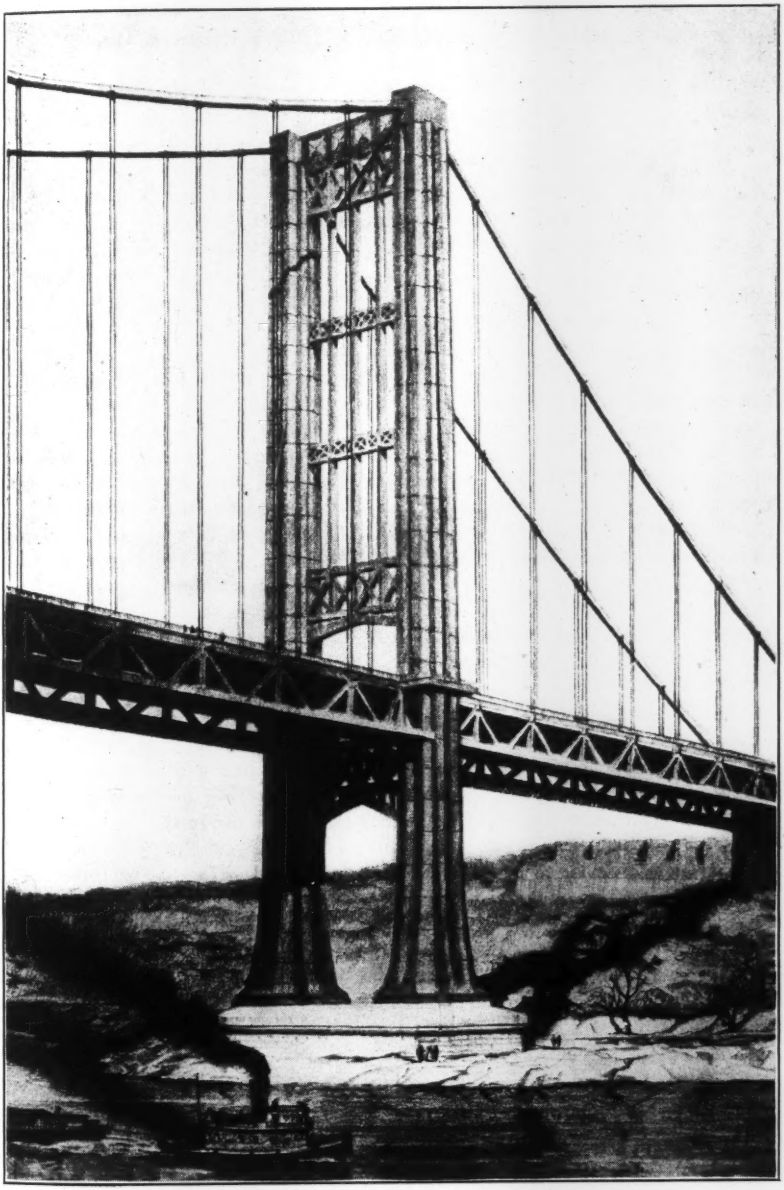


FIG. 21.—STUDY OF SLENDER TYPE OF STEEL TOWER.

Although no definite detail plans have been developed for the encasement, the towers have been the subject of extended architectural study. In their latest development in that respect (1931) they are illustrated in Fig. 22.

Since the steel tower frame is proportioned to carry the entire ultimate load for which the bridge is designed, the future addition of the concrete encasement, or possibly of a mere shell surrounding the steel frame, as has been suggested, or a combination of both, becomes essentially an æsthetic and architectural question. Before taking any action in this matter, the Port Authority will undoubtedly want to scan public opinion carefully, and particularly give due regard to the attitude of those civic and governmental organizations that are interested in the æsthetic development of the community and the preservation of the beauty of the landscape.

The writer, who has conceived and is primarily responsible for the type and general form of the design, considers the steel towers as they stand to represent as good a design as may be produced by a slender steel bent, and that they lend the entire structure a much more satisfactory appearance than he (and perhaps any one connected with the design), had anticipated. Nevertheless, he believes that the appearance of the towers would be materially enhanced by an encasement with an architectural treatment, such as that developed by the architect, Mr. Cass Gilbert, as illustrated in Fig. 22.

The writer is not impressed by the criticism, based solely on theoretical and utilitarian grounds, that the encasement would constitute a camouflage which would hide the true structure and its function. The covering of the steel frames does not alter or deny their purpose any more than the exterior walls and architectural trimmings destroy the function of the hidden steel skeleton of a modern skyscraper, except to the uninitiated.

Camouflage in this sense would condemn many of the creations in private and public life. It is an essential manifestation of civilization and is not incompatible with sincerity and honesty of endeavor, because an essential part of human effort is to create an æsthetic atmosphere, the value of which cannot be expressed in economic terms. This is evidenced in the craving for beautiful homes and public institutions which yields only to the limits of available means. Why should not a supreme effort be made in that respect in engineering structures, especially those which are viewed daily by thousands or millions of people?

Nevertheless if the encasement should not be built the writer will be satisfied that the effort to produce a massive structure has not been without fruits. The steel tower as it stands owes its good appearance largely to its sturdy proportions and the well-balanced distribution of steel in the columns and bracing.

At its present stage (1932) the top of the steel tower (Fig. 23) requires certain finishing additions. The cable bearings must be housed. It is also recognized that the tops of the towers more than 600 ft. above the water, which offer splendid views of the landscape for many miles all around, should be made accessible to the public by the provision of suitable elevators and protected observation platforms. The artist's sketch shown in Fig. 23

indicates the manner in which this may be accomplished by structural arrangement in harmony with the remainder of the tower frame.

As far as the engineering problem of designing a tower of this height—composed of a steel skeleton and concrete encasement with possible stone facing—is concerned, that presents a number of interesting phases which unquestionably require further research and intensive design study. However, such progress has been made with this composite type of construction, principally in connection with massive arches which exceed in proportions, magnitude, and complexity of stress action those of the proposed towers, that a satisfactory solution for the latter appears to be well within present possibilities. Joint action can be secured largely by proper structural bond between the steel members and the concrete, but probably the most important, and as yet insufficiently clarified, question involved, is that of the relative distribution of stress between steel and concrete and the elastic and non-elastic behavior of these materials under working loads, as well as close to the ultimate strength of the composite structure.

The Tower Steel Frame.—The design of the tower steel frame presented interesting and unusual phases. In its general proportions and outlines it was naturally governed by the design of the massive or encased towers, but an effort was made to design the steel frame so that it would present a neat appearance before its encasement.

In the structural arrangement it was found advantageous, largely on account of the enormous unprecedented load concentrations, to compose the steel structure of simple integral parts of relatively moderate size, but in such a manner as to assure a clear and certain, not necessarily statically determinate, stress action. An arch-shaped trussed frame, with a relatively large number of columns, offered itself in these respects as far more suitable than the usual, seemingly determinate, four-column tower with bracing in four planes.

As designed, the arched steel frame consists essentially of sixteen individual columns arranged in four arch-shaped transverse bents, each having two interior and two exterior columns. The exterior columns of each bent, and the bents themselves are slightly inclined, corresponding to the batter of the tower faces.

The cable bearings or saddles on the tops of the towers are above the two inner pairs of columns, but the load they transmit is distributed to all columns by cross-girders on top and the transverse bracing, in a manner which permits definite distribution of the load and practically exact proportioning of the various column sections. The design involved comparatively slight excess of metal in only the upper sections of the exterior columns.

That the criticism of eccentric loading on top with respect to some of the columns and the so-called static indeterminateness of the entire frame has no justification whatever has been demonstrated clearly by the elaborate stress investigations, supplemented by measurements on a celluloid model. Comprehensive stress measurements on the steel towers substantiated the assumptions and conclusions of the stress investigations.

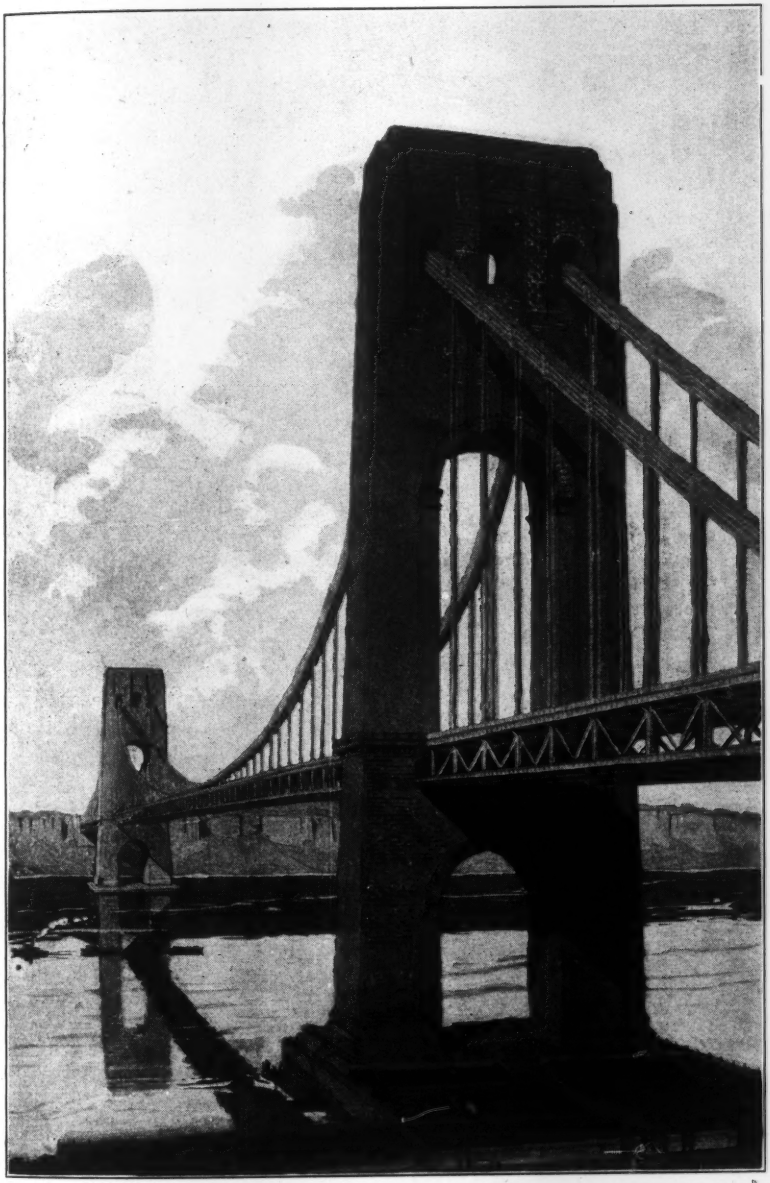


FIG. 22.—ARCHITECTURAL STUDY OF ENCASED STEEL TOWERS, WITH GRANITE FACING.

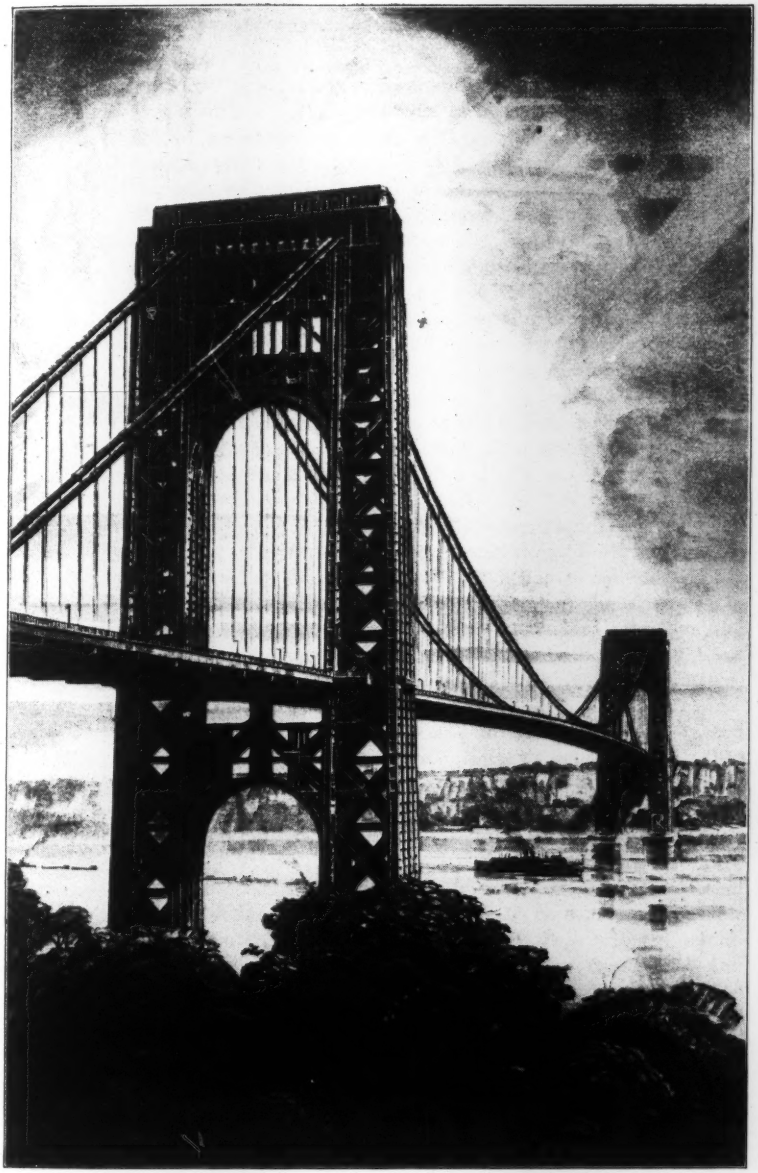


FIG. 23.—ARTIST'S SKETCH OF STEEL TOWER WITH PROTECTED OBSERVATION PLATFORM.

The Approaches and Highway Connections

Although structurally simple, the planning of the approaches of a large modern highway bridge in a developed community is one of the most intricate problems which such a project presents. Conceptions as to how such approaches should be planned—like those with respect to the planning of highways—have undergone radical changes as a result of the revolutionary change in the character, volume, and speed of highway traffic since the advent of the motor vehicle and, in this respect, engineers have not yet reached the stage of working with generally accepted principles and practices.

Moreover, the planning of approaches to such an important new crossing is intimately tied up with the planning of new, or the improvement of existing, highways leading to and from the crossing, in which many interests are involved. Consequently, the planning of the approaches to the George Washington Bridge was a task of extensive and intensive study and gradual evolution for a period of more than three years.

Under the statutes authorizing the Port Authority to build the bridge, provision is made for the approval of the approach plans by the municipalities in which the approaches are located, as well as by the Governors of the respective States.

As a practical procedure, the Port Authority sought and received close co-operation on the part of the City Government in New York and on the part of the local municipality, the Borough of Fort Lee, and the State Highway Commission in New Jersey, in the early development of the plans and in the necessarily lengthy negotiations which led to their final adoption and approval in the form of agreements between the Port Authority and these governmental bodies. In New Jersey, connections with county roads also involved dealings with the Bergen County authorities.

On both sides of the river the approaches as built are more elaborate and more efficient, as well as more costly, than was originally contemplated. The Port Authority did not shrink from assuming any reasonable additional expenditure for approach facilities as long as they were justified by increased efficiency of traffic distribution and satisfied the Municipal and State Governments.

As the planning proceeded it became apparent that certain principles to meet the requirements of modern traffic would have to be set up and, as far as reasonably practicable, followed. Foremost among these was the avoidance of crossing of traffic lanes at grade, not only on the bridge and approaches proper, but at the points of convergence and divergence of bridge and street traffic, because such crossings invite accidents and retard flow of traffic.

Likewise, primary importance was given to establishing direct connection of the bridge approach with a sufficient number of important streets or highways, which would permit of an adequate distribution of bridge traffic and would avoid excessive concentration in any one artery. Such a system of highway connections of course, had also to have ample flexibility to allow for the unavoidable fluctuations in the traffic flow. The old-time idea of a

single bridge plaza into which and from which all distribution takes place is no longer adapted to the modern requirements of speed and safety and must be superseded by a more efficient, but also more costly, system of direct roadway connections with the important highways carrying traffic to and from the crossing.

The adoption of conservative grades and curvatures and of ample width of approach roadways to secure greater safety and to permit more rapid flow of traffic also was considered essential.

The New Jersey Approach.—The general layout of the approaches is shown in Fig. 24. The roadway or upper floor of the bridge strikes the face of the Palisades at an elevation approximately 40 ft. below the top of the cliff; but the latter slopes down toward the west and meets the roadway grade about 600 ft. west of the face of the cliffs. Consequently, it became necessary to build the New Jersey Approach immediately west of the face of the cliffs in a rock cut.

Hudson Terrace, the most easterly north-and-south artery, lies in a depression and could be bridged over by the approach viaduct without material changes in its grade. It is connected, however, with the bridge approach both on its easterly and westerly sides by separate ramps. West of Hudson Terrace the main bridge ramp strikes the existing ground surface and widens out to form a plaza about 450 ft. long by 200 ft. wide, on which the toll facilities are located.

From this plaza roadway ramps ascend westerly to Lemoine Avenue (New Jersey State Highway Route No. S-1-A), while the central portion of the approach descends and passes under Lemoine Avenue and thence continues as a depressed roadway about 2 000 ft. to the west, where it connects with State Highways Nos. 1, 4, and 6, and a proposed county road. The plaza also permits direct connection with the local streets north and south and with Hudson Terrace to the east. The depressed roadway west of Lemoine Avenue also provides direct connection with Center Avenue, an important local artery to the west of Lemoine Avenue.

Thus, the bridge approach connects directly with seven more or less important through arteries and with a number of local streets, and all the important connections permit of uninterrupted flow of traffic without crossing of lanes at grade.

The New York Approach.—The New York Approach consists essentially of an elevated approach ramp from the anchorage in Fort Washington Park to a surface and a sub-surface plaza immediately west of Fort Washington Avenue, and of direct highway connections from there to Riverside Drive, Fort Washington Avenue, Broadway, and Amsterdam Avenue.

In accordance with the agreement between the City of New York and The Port of New York Authority, this approach was planned to be constructed in stages. In the initial stage of construction, the main approach ramp from the anchorage in Fort Washington Park is only 60 ft. wide and comes to grade at Northern Avenue. From there, side ramps connect easterly with Fort Washington Avenue, while the central roadway descends easterly to a

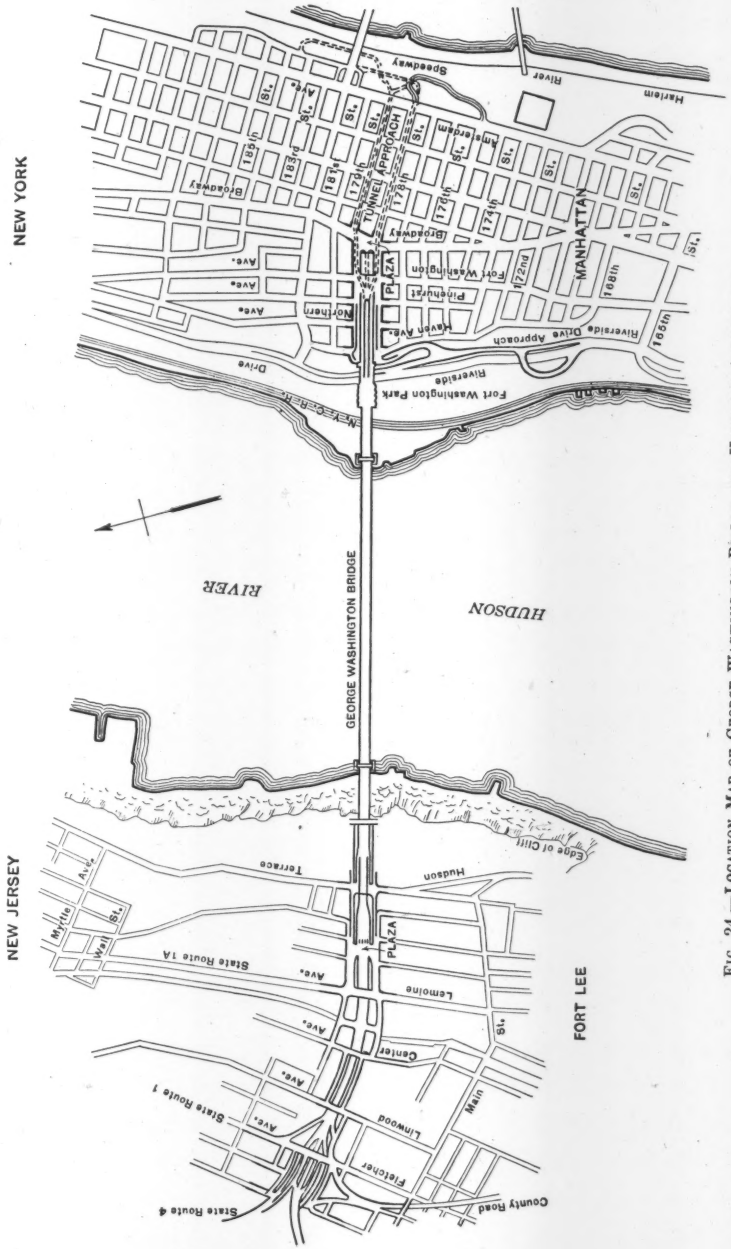


FIG. 24.—LOCATION MAP OF GEORGE WASHINGTON BRIDGE AND HIGHWAY APPROACHES.

sub-surface plaza west of Fort Washington Avenue, where connections are made with vehicular tunnels, each for two lanes, in West 178th and West 179th Streets.

In the initial plan the surface roadways in 178th and 179th Streets east of Fort Washington Avenue are widened to 36 ft. by reducing the width of the sidewalks in order to accommodate the bridge traffic which may go to, or come from, Broadway and adjacent streets. The initial plan also provides for the roadway connections with Riverside Drive, which run from Northern Avenue westerly parallel to 178th and 179th Streets north and south of the main approach ramp to Haven Avenue. West of Haven Avenue these roadways turn south, the on-bound roadway passing under the main approach ramp. Both the off-bound and on-bound roadways branch to permit connections on the east and the west side of Riverside Drive. The roadway for bridge traffic to and from the west side of the Drive is carried across it over an arch of 120-ft. span, approximately opposite West 171st Street. These connections permit trans-Hudson traffic to pass directly to and from Riverside Drive both northbound or southbound without crossing traffic on the Drive.

Toll Collection and Operation Facilities.—Owing to the necessity of collecting tolls, elaborate facilities therefor, as well as for police regulation, maintenance, and repairs had to be made. Property being much less expensive for such facilities on the New Jersey side and the concentration of these administrative activities being desirable, provision for them was made on the New Jersey side only, except in so far as traffic control, more particularly in the tunnels, may be necessary on the New York side.

Facilities for the collection of tolls are located on the spacious plaza formed by the widening of the bridge ramp west of Hudson Terrace and on the two side ramps connecting the main approach roadway with the east side of Hudson Terrace. The plaza width is sufficient to permit the use of sixteen lanes at the toll booths. These are arranged in pairs and are flanked on either side by a toll-house.

The dignified architectural treatment of the toll buildings is in keeping with the character of the bridge structure. This is secured by extending a continuous canopy over the line of toll booths terminating at the toll-houses. Aluminum surfacing is used on the booths while the toll-houses are two-story granite buildings of simple design.

The most advanced automatic equipment for the registration of vehicles and for the recording of collections is installed at all toll lanes. The type of vehicle and fare collected are recorded automatically and, at the same time, are indicated upon a dial system mounted at the toll booth. Beyond the toll booth a treadle records the passage of the car, thus providing an additional check. The pair of toll booths at the foot of each of the two ramps east of Hudson Terrace are similar in design and equipment to those at the plaza.

The adequate lighting of the plaza in the toll-collection area presented a special problem which was solved by the installation of four flood-light towers which provide illumination for 80 000 sq. ft. of plaza (200 ft. in width by 400 ft. in length), leaving the plaza entirely free of obstruction.

A field office and garage for the local administration of the bridge is located immediately south of the plaza. It is a two-story structure of colonial architecture with stone walls trimmed with granite. It provides suitable facilities for the Superintendent, his assistants, a maintenance supervisor, clerks, police officers, and a physician. It likewise provides locker space for the police and maintenance force, and space for store-rooms and a machine shop, the latter in connection with a garage of 16-car capacity which is attached to the field office.

Cost of Bridge and Approaches

At the time the bridge was financed in December, 1926, it was estimated that initially—ready for a four-lane vehicular traffic, but with provision as far as necessary for an ultimate capacity of eight vehicular lanes and four rapid transit tracks—the bridge would cost approximately $60 000 000. This included the highway connections on the New Jersey side to Hudson Terrace and Lemoine Avenue, and on the New York side the complete main approach for full capacity and connections with Fort Washington Avenue and Broadway, a total length of bridge and approaches of 8 720 ft.

As a result of negotiations with the City of New York there were subsequently added the highway connection with Riverside Drive, utilizing partly existing city streets, and the vehicular tunnel approach in 178th and 179th Streets between Amsterdam Avenue and Fort Washington Avenue. The 178th Street Tunnel was to be built initially and that in 179th Street later. In order to offset this additional initial expenditure partly, it was decided to omit, initially, the side ramps of the main approach west of Fort Washington Avenue and also the extension of the approach to Broadway. In part, the more costly additions were made feasible, without securing additional funds, owing to the fact that the estimates made in 1926 proved to be generally high, coupled with a decided lowering of costs in 1930 and 1931, as a result of the continued business depression.

The actual initial cost of the project, as now carried out and with ample allowance for the granite encasement of the New York anchorage, for the completion of the 178th Street Tunnel, completion of the top of the steel towers, and for miscellaneous other work yet to be done as part of the initial stage, is approximately, as follows:

Bridge Proper, Inclusive of Anchorages:	
Construction, engineering, and administration	$31 329 000
Real estate and easements	200 000
Total initial cost of bridge proper	$31 529 000
New York Approach, Inclusive of Connections with Amsterdam Avenue and Riverside Drive:	
Construction, engineering, and administration	$6 372 000
Real estate and easements	8 882 000
Total initial cost of New York Approach and highway connections	$15 254 000

New Jersey Approach, Inclusive of Connections with Hudson Terrace and Lemoine Avenue:	
Construction, engineering, and administration	$2 418 000
Real estate and easements	1 113 000
Total initial cost of New Jersey Approach and highway connections	$3 531 000
Summary of Cost of Bridge, Approaches, and Highway Connections:	
Construction, engineering, and administration	$40 119 000
Real estate and easements	10 195 000
Total construction and rights of way	$50 314 000
Interest during construction	4 543 000
Total initial cost of project, inclusive of interest during construction	$54 857 000

In the comparison of this cost with the par value of the outstanding bonds ($50 000 000) plus the advances by the two States ($10 000 000), there must be added to the cost an item of $3 030 000 for discount on the bonds, bringing the total initial cost of the project to $57 887 000.

In the original $60 000 000 estimate of 1926, the discount on bonds was not included, it having been assumed that any discount or premium would be a matter of financing operation, depending largely on market conditions and the interest rate on the bonds, and might properly be distributed over the period of amortization of the bonds. However, it was decided later to be desirable to amortize this item entirely out of construction funds.

The foregoing costs of construction, engineering, and administration of the bridge proper is made up of the following principal items:

Cost of Bridge Proper:	
Pier foundations and borings	$1 275 000
Anchorages (exclusive of steel)	2 730 000
Anchorage steel	1 467 000
Steel towers	8 069 000
Cables and suspenders	10 726 000
Steel floor system (highway deck only)	2 679 000
Floor-slab, railings, and miscellaneous construction	957 000
Electrical equipment and installation	85 000
Miscellaneous and contingencies	450 000
Engineering and administration	2 891 000
Total initial cost of construction, engineering, and administration of bridge proper	$31 329 000

An estimate made to determine what the cost would have been in case the bridge had been designed and built with only one deck for a six-lane roadway and two sidewalks, without provision for rapid transit traffic shows that the reduction would have been $15 000 000 and the total initial cost without discount on bonds, therefore, approximately $40 000 000.

About 95% of this possible reduction in cost is due to provisions for the greater capacity in the bridge proper, and only about 5% for such provisions made in the approaches. The vehicular tunnel in 178th Street is included in the aforementioned cost for a six-lane bridge.

Progress and Principal Construction Contracts.— As hereinbefore mentioned, the first Port Authority bonds for this project were sold on December 9, 1926. It was at that time estimated that the bridge would be completed ready for vehicular traffic on or about July 1, 1932.

Final borings at the site of the New Jersey Tower were then already under way under a contract with the Osborne Drilling Company, and on May 19, 1927, the first construction contract was let.

The major construction contracts were signed in accordance with the following schedule:

May 19, 1927.—Foundation for the New Jersey Tower, Contractor, Silas B. Mason, Incorporated, approximately $1 059 000.

June 18, 1927.—Excavation for the New Jersey Anchorage and Approach, Contractor, Foley Brothers, Incorporated, approximately $930 000.

November 4, 1927.—Steel towers and floor system, Contractor, McClintic-Marshall Company, approximately $10 753 000.

November 4, 1927.—Cables, suspenders, and anchorage steel work, Contractor, John A. Roebling's Sons Company, approximately $12 193 000.

May 4, 1928.—New York Anchorage and Tower Foundation, Contractor, Arthur McMullen Company, approximately $1 088 000.

December 6, 1929.—Demolition and removal of buildings on site of the New York Approach, Contractor, Klosk Contracting Company, approximately $150 000.

July 3, 1930.—Miscellaneous construction for New Jersey Approach at Hudson Terrace, Contractor, George M. Brewster and Son, approximately $323 000.

August 7, 1930.—Main approach ramp of the New York Approach, Contractor, Cornell Contracting Corporation, approximately $870 000.

August 7, 1930.—Vehicular tunnel in West 178th Street of the New York Approach, Contractor, Cornell Contracting Corporation, approximately $2 126 000.

October 14, 1930.—Riverside Drive connection of the New York Approach, Contractor, William P. McGarry Company, approximately $1 200 000.

February 2, 1931.—Paving and miscellaneous construction of the New Jersey Approach, Contractor, George M. Brewster and Son, approximately $565 000.

March 20, 1931.—Paving, railings, and miscellaneous construction work for the Main Bridge and New York Anchorage, Contractor, Corbetta Concrete Corporation, approximately $495 000.

Miscellaneous contracts were let in 1931 for the field office building, toll booths, electrical installations, flood-light towers, building alterations, and final painting to the aggregate amount of approximately $603 000.

Progress of construction work was so favorable as to make it possible to open the bridge for vehicular traffic on October 25, 1931, or about eight months in advance of the original schedule.

The actual period of construction from the date of financing to the opening of the bridge to traffic, therefore, consumed about a month less than five years, and from the date of the first construction contract less than four and one-half years.

Formal ground-breaking ceremonies took place on September 21, 1927. The raising of the first foot-bridge rope into place was celebrated on July 9, 1929, and the opening of the bridge was preceded by elaborate ceremonies on October 24, 1931.

Organization and Personnel

The Port of New York Authority, which under mandate from the States of New York and New Jersey has built, owns, and operates the George Washington Bridge is composed of twelve Commissioners, six from each State, appointed by the respective Governors. The Commissioners are men of broad and varied experience in business and public affairs. They serve without compensation.

During the construction of the George Washington Bridge, the following acted, successively, as Chairmen of the Port Authority: Mr. Julian A. Gregory (November 19, 1924, to May 20, 1926), former Governor of New Jersey, George S. Silzer (May 27, 1926, to June 30, 1928), and Mr. John F. Galvin (July 12, 1928, to date). The other Commissioners in office during that period include: Messrs. Ira R. Crouse, Howard S. Cullman, George R. Dyer, Frank C. Ferguson, William C. Heppenheimer, George deB. Keim, John F. Murray, John J. Pulleyn, Schuyler N. Rice, Alexander J. Shamberg, Herbert K. Twitchell (deceased), and Joseph G. Wright. Mr. Shamberg had been one of the Commissioners of the former New York Interstate Bridge Commission (later, the New York State Bridge and Tunnel Commission), since its creation in 1906, and General Dyer had served in that Commission since 1909 and was its Chairman during the construction of the Holland Tunnel.

The executive and administrative functions, including the financial and real estate transactions, are centered in the General Manager. Throughout the period of construction this responsible office has been held by Mr. John E. Ramsey.

The many complicated and unprecedented legal and legislative matters have been in charge of the Legal Department of the Port Authority, headed by Mr. Julius Henry Cohen, as General Counsel.

The engineering matters have been attended to by the Engineering Department, under the direction of the writer as Chief Engineer. He had the very able assistance of the following principal members of his staff:

Edward W. Stearns, M. Am. Soc. C. E., as Assistant Chief Engineer, attended generally to administrative functions in the Department and the preparation of contracts and specifications. Allston Dana, M. Am. Soc. C. E., as Engineer of Design, has been in charge of the Design Division. Mr. John C. Evans, Terminal Engineer, attended to the general studies for

the approaches and highway connections. Montgomery B. Case, M. Am. Soc. C. E., Engineer of Construction, headed the Construction Division which made all field surveys and directed and supervised the work in the field.

The architectural studies were made by Cass Gilbert, Architect, in collaboration with the Staff.

The Chief Engineer also had the advice of the following Consulting Engineers: William H. Burr, Gen. George W. Goethals (deceased), Leon S. Moisseiff (on design), Daniel E. Moran (on foundations), Ole Singstad (on vehicular tunnel approach), and Lewis B. Stillwell (on electric installations), Members, Am. Soc. C. E., and Mr. Joseph B. Strauss.

Gustav Lindenthal, Hon. M. Am. Soc. C. E., as Consulting Engineer, rendered special advice on design questions.

Charles P. Berkey, M. Am. Soc. C. E., as Consulting Geologist, made the geological investigations.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 1819

GEORGE WASHINGTON BRIDGE: ORGANIZATION, CONSTRUCTION PROCEDURE, AND CONTRACT PROVISIONS

By Edward W. Stearns,[1] M. Am. Soc. C. E.

Synopsis

For the design and construction of the George Washington Bridge, simultaneously with other large bridge projects, The Port of New York Authority built up a large engineering organization. In its development, and also in arranging the program of the construction work, the basic aim was to secure flexibility, which would permit efficient handling of the complex problems involved and would insure rapid construction. This paper contains a review of the preliminary steps leading to the creation of the engineering organization, and describes briefly the organization and its method of carrying on the work, the construction program developed for the building of the George Washington Bridge, the procedure followed in handling construction contracts, and the outstanding provisions of the construction contracts and specifications.

Preliminary Steps

In the early part of 1925, the Legislatures of the States of New York and New Jersey passed concurrent legislation[2] authorizing The Port of New York Authority,

"To construct, operate, maintain and own a bridge, with the necessary approaches thereto, across the Hudson River from points between 170th Street and 185th Street, Borough of Manhattan, New York City, and points approximately opposite thereto in the Borough of Fort Lee, Bergen County, New Jersey."

By virtue of this legislation the State of New York appropriated $100 000 and the State of New Jersey $150 000 for the preliminary work necessary

[1] Asst. Chf. Engr., The Port of New York Authority, New York, N. Y.

[2] Chapter 41, Laws of New Jersey, 1925; Chapter 211, Laws of New York, 1925.

for making borings, surveys, engineering studies, investigations, and for hearings, and all expenses incidental to these activities. This legislation provides that the sums appropriated shall be repaid to the States when the cost of construction has been fully paid for and the debt or debts created for such purpose has been amortized. Both Acts provided, however, that no money should be expended by the Port Authority until equivalent amounts had been appropriated by both States. Accordingly, later in 1925, the State of New Jersey furnished $100 000 out of its total appropriation of $150 000, equivalent to that appropriated by the State of New York. An amount of $200 000 was thus made available to the Port Authority for its preliminary studies.

The Port Authority began its studies immediately and, on March 11, 1926, practically one year after the legislative authorization for the project, transmitted a report[8] to the Governors of the two States. The work accomplished to that date included: (*a*) Comprehensive studies to determine the probable volume of traffic over the bridge and the revenues to be derived; (*b*) topographical surveys, river borings; (*c*) engineering design studies to determine the suitable site, size, and type of crossing and its cost; and finally, (*d*) architectural studies to determine the feasibility of "rendering the bridge a befitting object in a charming landscape." The report set forth eleven conclusions, which may be summarized as follows:

(1) The traffic studies revealed an urgent demand for a crossing for vehicular traffic in the vicinity defined by the legislation, and indicated that the traffic would be of sufficient magnitude to make the undertaking financially feasible.

(2) The general location was well chosen both in regard to topography and feasibility of convenient connections to important local and arterial highway routes.

(3) From the engineering point of view, the construction of the bridge with a single river span of at least 3 500 ft. and a clear height above water of about 200 ft. was feasible in every respect and would involve no extraordinary difficulties, nor hazardous or untried operations.

(4) The suspension bridge would be the most economical type and æsthetically superior to any other type.

(5) Should funds for the construction of the bridge be available in 1927, it was expected that, not later than 1933, the bridge would be open for 4-lane vehicular and bus passenger traffic and for pedestrians.

(6) On the basis of the information available prior to completion of the preliminary studies it was estimated that the bridge could be constructed in an initial stage, and opened to highway traffic at a cost of $50 000 000.

(7) Depending upon the traffic capacity finally to be decided upon, the bridge could be enlarged later at an additional cost of between $15 000 000 and $25 000 000.

(8) Conservative traffic analysis indicated that the bridge would be self-sustaining in every respect from the first year, without unreasonable toll charges on traffic.

[8] Tentative Report of Bridge Engineer on Hudson River Bridge at New York between Fort Washington and Fort Lee.

(9) Growth of vehicular traffic might justify enlarging the bridge, within ten years after its opening, to 8-lane capacity, and within twenty years thereafter, the entire bond issues for construction cost might be amortized.

(10) Architectural studies indicated that the bridge could be designed to blend harmoniously with the grandeur of its natural setting.

(11) On account of the favorable aspect of the bridge and its urgent necessity, it was recommended that preliminary work be carried to completion, and that the States be asked to appropriate an additional sum of $100 000 for that purpose.

Subsequent to the submission of this report, the State of New York made an extra $50 000 appropriation; the additional $50 000 was provided out of the authorized total appropriation of $150 000 in the original legislation by the State of New Jersey. This brought the funds for preliminary studies to the total of $300 000. With these funds it was possible to conclude the preliminary studies before it became necessary to approach the bankers for financing the construction work. In order to provide for financing the bridge construction, the two States enacted additional legislation[4] which was approved by the Governors on March 10, 1926, and May 4, 1926, respectively. These two Acts, subject to certain limitations, pledged the sum of $5 000 000 from each State, payable in five annual installments of $1 000 000. The money, with interest as and when earned, is to be repaid to the States out of revenues derived from tolls, after operating expenses and debt charges incurred for the construction of the bridge have been met. The legislation further provided that the remainder of the money needed for construction and incidental purposes was to be raised by the Port Authority on its own obligations, secured by the pledge of the revenues and tolls arising out of the use of the bridge. The obligation for moneys so raised constitutes a prior lien on the revenues and tolls.

On December 9, 1926, the Port Authority concluded negotiations with bankers for the sale of $20 000 000 of bonds for construction purposes, out of a total authorized issue of $60 000 000. A further installment of $30 000 000 was sold in the spring of 1930 to complete the financing. The monies advanced by the two States, with the $50 000 000 derived from the sale of Port Authority bonds, made the total of $60 000 000. The greater part of this sum has been used to bring the project to its present (1932) stage of completion.

Engineering Organization

In 1924, the two States had directed the Port Authority to make preliminary studies for, and to undertake the construction of, two bridges over the Arthur Kill—one between Elizabeth, N. J., and Howland Hook, Staten Island, New York, and the other between Perth Amboy, N. J., and Tottenville, Staten Island. The Port Authority retained Waddell and Hardesty, Consulting Engineers, to make the preliminary design studies for these bridges, under the direction of Mr. W. W. Drinker, then Chief Engineer of the Port Authority. In 1925, it became evident that, for making the pre-

[4] Chapter 6, Laws of New Jersey, 1926, and Chapter 761, Laws of New York, 1926.

liminary studies for the George Washington Bridge, for directing and supervising the construction of the two bridges over the Arthur Kill, and for the handling of the possible fourth project of a bridge over the Kill van Kull between Bayonne, N. J., and Staten Island, a bridge engineering organization of its own would be advantageous to the Port Authority. Accordingly, O. H. Ammann, M. Am. Soc. C. E., later Chief Engineer of the Port Authority, was employed as Bridge Engineer, and he began immediately the building of an engineering staff. A plan of organization was developed, the soundness of which is attested by the fact that, considerably amplified, it stands to-day, after having successfully accomplished the purposes for which it was developed in 1927. It consisted essentially in separating the engineering work of the Bridge Department into divisions, placing at the head of each division an engineer well qualified by training and experience to handle his particular work. Responsibility was placed upon each division engineer for producing work from that division and co-operating with the heads of the other divisions under the general direction of the Bridge Engineer.

The work was divided into the following five divisions: Traffic Studies, Design, Contracts and Specifications, Construction, and Planning of Approaches and Highway Connections. Selection of the personnel for these separate divisions was made chronologically as the need developed.

Traffic Studies.—The Port Authority has no power of taxation, has no authority to assess for benefit, and, at the time of financing the bridge, it owned no physical properties which in themselves could be used as collateral for loans. All the moneys which it borrowed had to be protected by the revenues to be derived from the tolls. It was essential, therefore, for the Port Authority to make a most careful survey of traffic conditions and from this survey to draw conclusions that could stand the acid test of the inquiries of prospective investors.

The Division for Traffic Studies, including a force of traffic inspectors and analysts, was in charge of the Traffic Engineer. As a rule the inspectors were employed only temporarily for such periods of time and at such particular locations as was necessary to accumulate the needed data. The analysts assembled these data and from them forcasted the traffic that would make use of the facility over a period of years.

The studies involved a comprehensive analysis of the following factors:

(*a*) The volume of vehicular and pedestrian traffic over each of the seventeen Hudson River ferries between the Battery, New York City, and Tarrytown, N. Y.

(*b*) The volume of traffic that could reasonably be expected to be diverted to the bridge from each of these crossings.

(*c*) The volume of new traffic that the bridge could be expected to attract, which involved an estimate of the probable effect of the opening of the Holland Tunnel.

(*d*) The total volume of probable traffic over the bridge for each year for a period of twenty years subsequent to its opening, including considera-

tion of probable effect upon bridge traffic of the possible construction of other new crossings south of 179th Street.

(*e*) The estimation of probable revenues for each year of the 20-year period.

In order to determine the probable volume of traffic which would be diverted from existing ferry crossings, it was necessary to ascertain the distribution of the traffic over each of the ferries, by finding the origin and destination of each vehicle for sample periods of time so selected that the peak and average traffic conditions were reflected. The peak condition occurs in July and the average condition in October. Variations of traffic between week days and Sundays, and from hour to hour, had also to be considered. Inspectors were placed on each of the ferry-boats throughout the day, and they recorded the type of vehicle (that is, whether horse-drawn or motor-propelled, and whether commercial or pleasure vehicle). They also recorded the carrying capacity of the various vehicles, the number of persons actually carried in each vehicle, the State license, the origin and destination of each vehicle, and the frequency with which the particular vehicle used the particular ferry route. These "clockings" by inspectors were made throughout the months of July, August, September, and October, 1925, a force of fifty-six men being employed on the seventeen ferry routes. Detailed information was recorded for a total of 242 000 vehicles.

Clockings were also made of the traffic passing over the streets and highways at advantageous points, not only in the vicinity of the bridge site, but also at points considerably distant from the bridge, in order to determine the potential capacities of these highways for bridge traffic, because the degree of resultant congestion on these arterial connections would affect materially the flow of traffic to the bridge. These investigations included also a comparative study of the traffic carried by the East River bridges particularly during the peaks of traffic. The records kept by each of the seventeen Hudson River ferries were made available and were investigated as far back as 1914 so that an excellent record of the total river crossing was subject to analysis for a considerable period. These records served to show also the periods of peak traffic and were very valuable in forecasting the probable future traffic, under the improved conditions, which the opening of the Holland Tunnel and the construction of the bridge would create. In connection with these studies, consideration was also given to a comparison of the growth in motor-vehicle registration in the Metropolitan District with the flow of the traffic over the river. This study showed clearly that the inconvenience and insufficiency of the facilities provided by the ferry companies acted as a restriction on the flow of trans-river traffic and led to the conclusion that the provision of more convenient facilities would materially increase this traffic. The accuracy of this observation was conclusively demonstrated after the opening of the Holland Tunnel. Detailed figures of the forecasts derived from these various traffic studies have been presented[5] by Mr. Ammann.

Because of the importance of the traffic studies as a basis for further vehicular crossings, they have been continued by the Port Authority. As a

[5] See p. 16.

result of the information tabulated during the seven years, 1925 to 1932, it is possible to predict not only the future volume of traffic, but also the effect of additional crossings.

A change in the organization for traffic studies was made in 1930, when this division of the work was transferred from the Engineering Department to the Bureau of Commerce of the Port Authority.

Design Division.—Centered in the Design Division was the work of preparing the preliminary design studies, general drawings and layouts, stress calculations and design, detailed contract drawings, estimates of cost, the checking of contractors' shop and working drawings, and work of a similar nature. In order to co-ordinate properly the work which was performed under the various contracts, particular attention had to be given to the preliminary design studies, general drawings and layouts, and to the estimates of cost, so that the work that had not been placed under contract would fit properly with that which had already been placed under contract, and also so that the cost of the entire project would not exceed the estimates upon which the financing was based. The preliminary designs were frequently modified and the estimates of cost revised accordingly. The contract drawings were usually elaborate in their details and although they were frequently revised as the work progressed, nevertheless, they clearly covered the general character of the work to be performed.

The Design Division was subdivided into two general parts, based on the character of the work to be done. Reporting directly to the Engineer of Design, who was in general charge of the Division, were the Assistant Engineer of Design and the Chief Draftsman. All the general design studies, stress calculations, cost estimates, etc., were placed under the direction of the Assistant Engineer of Design. The preparation of contract drawings, detail studies, layouts, and work incidental thereto was placed under the direction of the Chief Draftsman. The sub-divisions were quite flexible, and men were continually transferred from one sub-division to the other, as the need for the Staff Personnel varied, and not infrequently, when it was convenient to the handling of particular problems, men were working for both subdivisions at the same time.

Directly subordinate to the Assistant Engineer of Design and the Chief Draftsman were a number of Assistant Engineers, each of whom was especially qualified in some particular phase of work—structural steel design, reinforced concrete design, foundations, highway construction, etc. These men functioned somewhat as "squad bosses" do in usual drafting-room practice; they also superintended the work of groups of individuals. Depending on the work in hand, the individual groups, at any one time, might be engaged on work of similar nature on all the bridge projects, or only on one project, and because of the shifting of men from group to group as occasion demanded, the groups varied materially in number from time to time. Although the effort was made to keep each man engaged in the particular work for which he was best qualified, this specialization was not considered all-important, and through the transference from one group to another, each man was given as wide a variety of work as possible. This policy served to keep up a high

morale among the men; it also gave each man a wider and more intimate knowledge of each structure as a whole, and thus led to more accurate and intelligent solution of the problems in hand. It is recognized, of course, that this form of organization necessitated a higher proportion of supervision and direction but, against this, was balanced a better co-ordination of the various parts of the work with a resultant higher degree of efficiency.

Central Division.—The Central Division, in charge of the Assistant Chief Engineer, who reported to the Chief Engineer, was organized to handle the preparation of specifications and contracts, reports, and general administrative matters of the Engineering Department. Negotiations leading up to, and the preparation of agreements with the various outside bodies whose interests were allied with, or affected by, the construction of the bridge, were handled by this Division.

Flexibility of organization was also essential in the Central Division, because it was necessary to co-ordinate to a considerable degree the functions of the various Divisions in the Engineering Department and to contact with the other Departments of the Port Authority Staff, such as the Legal Department, Real Estate Department, Treasurer, and Auditor. Routine administrative matters were in charge of an Assistant Engineer who reported to the Assistant Chief Engineer. The preparation of contracts and specifications was placed under the direction of another Assistant Engineer, chosen for his particular experience along these lines. During the preparation of the contracts, close contact was maintained not only with the Division heads, but with the Assistant Engineers engaged in the work in the Design and Construction Divisions, and with those engaged in the inspection of materials. Specifications were invariably prepared in draft form and in sufficient number to submit to all those interested, who were thus given an opportunity to review not only the parts of particular interest to them, but also the entire specification. This policy brought about a more complete understanding of each other's problems as between the men in the office and those engaged on construction in the field; it resulted in broader, more complete, and more comprehensive specifications, and minimized discrepancies and omissions between the drawings and specifications.

Progress pictures of the work received special consideration. A photographer was employed on the staff of the Central Division, and full equipment for him was provided as soon as construction work began. He reported, at more or less regular intervals, to the Resident Engineers, and received instructions from them as to what particular operations should be photographed. This policy resulted in producing a complete pictorial record, which the writer feels was thoroughly justified by the magnitude and importance of the work.

Closely allied to the photographic progress record, although handled as an entirely separate entity, is a film history of the construction of the main span of the bridge. These films show the actual construction in the field of the principal operations and have proved valuable in acquainting the public with the progress of the work and the methods used. They constitute a record that should prove of great interest and value to succeeding generations.

Construction Division.—All the preliminary surveys, including topography, triangulations, river soundings, mapping, and supervision of subsurface borings, and, later, the detailed surveys necessary for the preparation of the contract drawings, were made by the Construction Division. This Division also had charge of the supervision and inspection of the field work, including the establishment of the lines and grades upon which the various parts of the structure were constructed, but excluding the inspection of materials. The measurement of the work performed and the preparation of the approximate monthly estimates and the final estimates, upon which the contractor's total compensation was based, also originated in this Division.

After careful analysis of the problems involved in the construction of the George Washington Bridge, the Construction Division, in charge of the Engineer of Construction, was subdivided into three complete and independent sections—each headed by a Resident Engineer who reported to the Engineer of Construction. The division into sections was effected geographically: The New York Section had charge of all work on the New York side of the river; the New Jersey Section had charge similarly of work on the New Jersey side; and the Central Section had charge of the work of finishing the two top panels of each tower and all work on the suspended structure, floor system, roadway, and appurtenances in connection with the bridge proper. The anchorages themselves were under the jurisdiction of the Resident Engineers of the New York and New Jersey Sections, respectively, but all the work on the cables, the floor system, and the roadway, from the easterly end of the New York anchorage to the westerly end of the New Jersey anchorage, was assigned to the Central Section. These various Sections maintained their entities until the bridge was nearly completed, when the Central and New Jersey Sections were merged under the Resident Engineer of the Central Section, in order to release the Resident Engineer of the New Jersey Section for work on surveys of the proposed Midtown Hudson Tunnel.

Only in the case of the erection of the steel towers (both of which were part of a single contract), was there any overlapping of functions between the Sections. The Resident Engineers of the New York and New Jersey Sections supervised the erection of the towers on their sides of the river, the Central Division not being established until after the completion of the first ten panels. No difficulty was involved in this overlapping, however, since the contractor elected to erect both towers simultaneously, with entirely separate equipment and gangs of workmen. As a result, a friendly rivalry developed, not only among the contractor's forces, but also among the Port Authority engineers and inspectors, which was a distinct advantage in producing good workmanship and rapid progress.

The personnel of the Construction Division varied considerably in number, depending on the work under way, but during the greater part of the period of construction, from fifty to seventy-five engineers and inspectors were employed at the bridge site.

Approach Studies.—A special division of the Engineering Department was organized at the beginning of 1929 to handle the general planning of the approaches and the highway connections to both the George Washington Bridge and the Bayonne Bridge. This Division consisted of the Terminal Engineer, who reported to the Chief Engineer, an assistant to the Terminal Engineer, and a number of draftsmen. Comprehensive studies of the traffic needs at the terminals of these bridges were made in close co-operation with the engineering representatives of the municipalities and State organizations that were affected. These studies led to the adoption and approval of approach plans satisfactory to all the agencies concerned, and these plans formed the basis for the agreements with the municipal and State bodies.

In addition to the five major sub-divisions of the Engineering Department, two sub-divisions were created to handle particular work, and, later, these were placed under the general supervision of the other principal divisions. The organization of these sub-divisions as separate units, and their subsequent merging with the other divisions is indicative of the general flexibility in the plan of organization.

Inspection of Materials.—Inspection of all materials used in the Arthur Kill Bridges was placed under the direction of an Engineer of Inspection who built up an efficient organization of masonry material inspectors and metallic material inspectors. A small laboratory, commensurate with the relatively small volume of work involved, was established in rented quarters in Jersey City, N. J., and this organization and equipment proved satisfactory as long as only the Arthur Kill Bridges were under construction. With the start of the work on the George Washington Bridge, however, it soon developed that not only were the facilities insufficient, but that better results could be achieved by separating the inspection of metallic materials entirely from the inspection of masonry materials. Accordingly, two sub-divisions were formed: One devoted to inspection of concrete materials, brick, masonry, etc., under the Engineer of Masonry Inspection; and the other devoted to the inspection of metallic materials of all kinds, under the direction of the Engineer of Steel Inspection. At first, these two came under the direction of the Chief Engineer. In 1929, this arrangement was modified so that the Engineer of Masonry Inspection was placed under the jurisdiction of the Assistant Chief Engineer, and the Engineer of Steel Inspection under the Engineer of Design. This modification was for purposes of organization only, and in no way affected the functional duties of either sub-division.

Each of the Engineers of Inspection had inspectors reporting to him. Those inspecting materials in the New York District were stationed at the laboratory in Jersey City. Others were stationed at the various plants where materials were being manufactured or fabricated. In this way a thorough and complete check of the quality of all materials entering into the bridges was maintained. Only the best quality of material and the highest grade of workmanship were permitted throughout the entire work, and due credit should be given to the contractors who contributed whole-heartedly toward maintaining this standard.

It seems proper, at this point, to describe the modern testing laboratory constructed in Jersey City in 1929, when the less completely equipped and smaller laboratory became too congested for further efficient work. Although, of course, it is not a part of the George Washington Bridge organization, as such, its functions are closely allied to those of the sub-divisions for the inspection of materials.

The Port Authority Laboratory building is 50 by 100 ft. in plan. It consists of two stories and basement, with a higher portion on one end to provide housing for a 1 000 000-lb. testing machine. The building is of structural steel, on concrete foundations, and has brick walls backed with tile, concrete floors, and tile partitions.

It contains facilities for routine specification acceptance tests for concrete and for all the check tests on structural steel, and it also has facilities for conducting some research work. In the main testing room, in addition to the 1 000 000-lb. machine, is a 50 000-lb. testing machine that is used for smaller concrete test specimens and for machine tests for structural steel and castings. A view of this equipment is shown in Fig. 1. This main testing room is arranged so that trucks can be driven into it from the street, in order that heavy pieces may be lifted from them by an overhead traveling crane. In addition to the main testing room, there is a machine shop, a chemical laboratory, and concrete, cement, and aggregate laboratories. Fig. 2 shows a general view of the chemical laboratory. In the basement there is a moist room for curing concrete specimens, with equipment for maintaining constant temperature and humidity. The building also contains office space, drafting-room, and inspectors' rooms.

Research and Tests.—Because of the unusual magnitude of the undertakings on which the Port Authority has been engaged, it was deemed advisable to establish a Division of Research, the purpose of which was to make scientific investigation of important problems on which it was felt that there might be a lack of published information. It is not the purpose of this paper to treat of the work done in this Division, since it will be described in other papers.

In brief, the functions of this Division were the making of stress measurements in the towers and in the anchorage steel of the George Washington Bridge, the investigation of full-sized specimens representative of certain tower sections, and of certain sections of the Bayonne arch, and the making of stress measurements on the Bayonne arch. Some of the work of this Division was done in co-operation with the National Bureau of Standards, in Washington, D. C. This Division was headed by an Engineer of Research and Tests, who at first reported directly to the Chief Engineer, and, later, to the Engineer of Design. Under the Engineer of Research and Tests was a group of assistants chosen particularly for their interest in research matters. These men not only carried on the actual work, but developed lines of investigation particularly suited to the purposes for which the Division was organized. They designed, and in many cases made, special instruments for obtaining the results they were seeking.

Program of Construction

After comprehensive study of possible methods, it was decided to divide the construction work into a number of separate contracts, the plans and specifications for each such part being prepared as the progress of the work as a whole required. In so far as separation between the main span and approaches was involved, division was quite essential in any event. The State laws authorizing the construction of the bridge provided that "the plan of the approaches at either end of the bridge shall be subject to the approval of the respective Governors of the States of New York and New Jersey and of the respective municipalities in which they shall be located." Such complete and comprehensive studies were necessary for the solution of the approach problems that the construction of the main span would have been delayed several years had it been necessary to defer it until the approach plans had been definitely settled. It was deemed advisable, however, not only to make a separation of the work as between the approaches and the main bridge, but to let separate contracts for the various integral parts of the structures as the work progressed.

The advantages of this method of procedure are, briefly, as follows:

First.—In such a large project it is essential that an extended study of the various parts be continued after the project has been financed. Thus, study of certain parts can be deferred with advantage and, if conditions arise that make it necessary or desirable, certain parts can be submitted to restudy and possibly changed more or less radically.

Second.—By preparing detailed contract drawings and specifications as the work progresses, more thorough and elaborate treatment can be given to these subjects, and, at the same time, much more rapid progress can be made in construction because certain parts of the work can be started prior to the elaboration and completion of studies on the entire project. An actual example of these advantages is given by the case of the New Jersey anchorage and the contract for the bridge cables. At the time bids were received for the New Jersey anchorage final decision had not been made either as to whether the cables should be of eye-bar chains or steel wires, or, if wire cables were used, whether each pair of cables would have one cable placed above the other, or would have the cables side by side. Upon decision of these questions depended the ultimate dimensions of the anchorage tunnels and pits. However, it was anticipated that, by the time the contractor for the anchorage was ready to start excavation of the anchorage tunnels, decision on these questions about the cables would have been reached. It was possible, therefore, to show, on the contract drawings, the average dimensions for the tunnels, upon which the contractor could base his unit price for the excavation. This permitted him to finish much of his work before the final determination of dimensions had been reached. The anchorage work was thus begun at a sufficiently early date to expedite construction of the bridge as a whole.

Third.—This procedure eliminates the difficult and uncertain task of developing a definite time schedule for all parts of the work at the outset and, at the same time, permits taking advantage of any saving of time effected on

FIG. 1.—VIEW OF LABORATORY BUILDING, SHOWING 50 000-POUND TESTING MACHINE.

FIG. 2.—VIEW OF CHEMICAL LABORATORY, JERSEY CITY, N. J.

particular parts of the work. It avoids any conflicts and litigation that might arise because of delays.

Fourth.—By avoiding the need of awarding contracts for construction work, the performance of which could not be begun for several years thereafter, considerable economy is possible through relieving bidders from the necessity of protecting themselves against uncertain changes in prices of materials or labor that may develop during the intervening period.

Fifth.—The award of contracts for the separate parts of the work left more time available for the acquisition of the necessary real estate. This additional time permitted negotiations for more favorable purchase prices, and, in the case of improved real estate, permitted its continued use subsequent to purchase and up to the time it was necessary for it to be turned over to the contractors, for producing revenues and thus lightening the total financial burden.

As soon as the financing of the project was assured, a careful study was initiated to devise a program of construction that would permit the fullest competition in bidding, would result in the most logical sub-divisions of the various elements of the work, and would occasion no delay in the starting of various parts, or in the progress of the work as a whole. The divisions of the work, and the construction contracts for each division, were arranged in logical classification as to nature and location. For example, the New Jersey tower required deep foundations, entirely different in character from those for the New York tower, whereas the New York tower foundations, both in the nature of the work and in geographical location, were closely allied to the work for the New York anchorage. Accordingly, one contract covered the New Jersey tower foundation work, whereas another combined in a single contract the New York anchorage and the New York tower foundations. Again, the New Jersey approach and the New Jersey anchorage both required the excavation of rock in the Palisades and could be included in a single contract. The steelwork for the bridge proper could be handled either as a single contract or as two or more contracts. The work on the approaches could logically be subdivided into contracts classified as to its nature and location.

At first, the studies of classification were made only for the earlier and major elements of the work, the schedule of the later operations being deferred until the details of design could be given further study and until more accurate determination could be made of the work involved. This deferring of problems that did not require immediate attention made more time available for careful analysis and solution of those involved in the earlier work. After determining the classification into contracts, attention was given to an analysis of the major operations involved in each contract, in order to allow the proper period of time for their completion. Finally, consideration of the problem as a whole led to determination of the approximate date at which each contract would have to be started in order not only to have it begin immediately upon completion of the next preceding contract, but also to have it completed by such date as to provide ample time for the next succeeding

contract. These studies led to the first construction program, which was adopted on April 1, 1927, although previous to that time, several tentative schedules had been prepared for the purpose of arranging the financial program, and determining the probable date for completion of the entire project (see Fig. 3).

The construction program had an important function to perform as applied to the work of the Port Authority Staff. Since it fixed the approximate date on which each contract should be started, it was possible to work backward from this schedule and determine the time when the drawings for any particular contract would have to be started and completed, and when the drafts of the specifications would have to be distributed to the Staff to allow sufficient time for proper study. In short, it became a schedule that had a direct bearing on all the operations in the Engineering Department. It was used in preparing the necessary forecasts of financial requirements, for determining the dates on which certain parcels of property would be required, and for other more or less similar functions.

So many conditions affect the actual time required to complete a particular piece of work that it is, of course, impossible to predict exactly when the work will be finished. Therefore, the program itself, was properly regarded as something flexible, although it was developed with the various elements intermeshed into a complete whole. It was planned so that adjustments could be made from time to time to compensate for work completed sooner or later than anticipated. It is only through this flexibility and constant vigilance that full advantage can be realized from any predetermined construction program. It will be noted from the program of April 1, 1927 (Fig. 3), that the order of construction was as follows: New Jersey tower foundations, excavation for the New Jersey anchorage and approach, steelwork for the main structure, foundations for the New York tower, and, finally, masonry for the first stage of the New York anchorage.

Modifications of this program illustrate the flexibility in its actual handling. Even before the end of 1927, only the first two contracts remained in their original order. The New York tower foundations and the New York anchorage were combined into a single contract, which at first was planned to precede the awarding of the contract for the steelwork of the main bridge. Because of delay in the final determination of the placing of the New York anchorage in Fort Washington Park, a further modification of the program was made, so that the award of the steel contract preceded, by several months, the award of the contract for the New York anchorage and tower foundations.

As a result of the necessity for flexibility, the entire program was kept at first in a state of flux, changes being made as necessary to meet delays and varying conditions which invariably occur at the beginning of such large undertakings. It was felt inadvisable under these circumstances to issue new programs each time a change was made, and, therefore, the changes were noted by the individuals on their own copies of the program in any

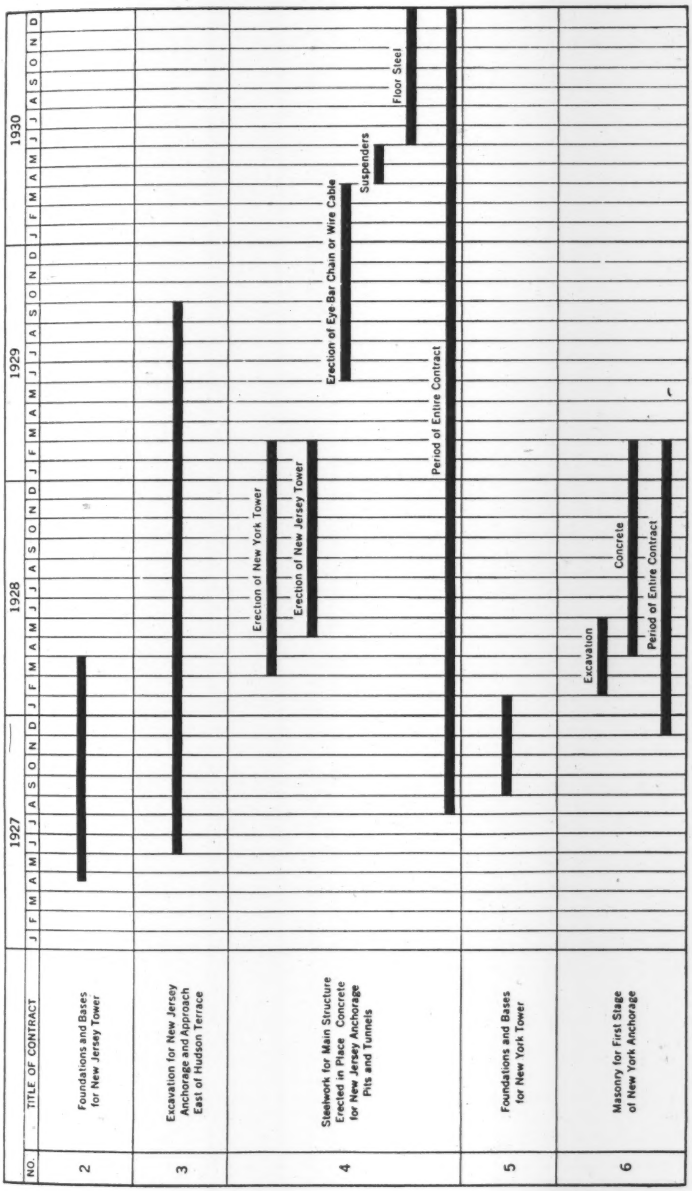

NO.
TITLE OF CONTRACT
1927
1928
1929
1930
J F M A M J J A S O N D J F M A M J J A S O N D J F M A M J J A S O N D
2
Foundations and Bases for New Jersey Tower
3
Excavation for New Jersey Anchorage and Approach East of Hudson Terrace
4
Steelwork for Main Structure Erected in Place Concrete for New Jersey Anchorage Pits and Tunnels
Erection of New York Tower
Erection of New Jersey Tower
Erection of Eye Bar Chain or Wire Cable
Suspenders
Floor Steel
Period of Entire Contract
5
Foundations and Bases for New York Tower
6
Masonry for First Stage of New York Anchorage
Excavation
Concrete
Period of Entire Contract

FIG. 3.—FIRST CONSTRUCTION PROGRAM.

way best suited to their own needs. On May 5, 1928, a second program was formally prepared and issued, but, here again, because of the necessity for flexibility, the program remained in force for only nine months, or until February 5, 1929, when a further revised program was prepared. In these programs the principal operations in connection with the various contracts (all of which had an important bearing upon the subsequent work) were shown in detail.

It is needless to go into more detailed discussion of all the changes made, or the reasons for them. Enough has been said to illustrate the extreme flexibility of the schedules and to show their value in connection with work of this character. An attempt has been made, however, to illustrate graphically the fluctuations in the construction programs by the preparation of a composite chart (Fig. 4) on which the solid lines show each part of the work as it was actually done, and, the heavy dashed lines, the extreme variations as set up on different construction programs. In the extreme left-hand column of this diagram the contract numbers are given, which originally were planned to run consecutively as the contracts were awarded, but which, as the diagram shows, did not follow consecutively in each case. For some contracts, two or three numbers are given in the chart. The upper number is invariably the actual contract number, and the lower number, or numbers, are contract numbers which at one time or another were assigned to the work in successive construction programs.

Because of the necessity of controlling certain operations under individual construction contracts, in addition to controlling the time for completion of the work, the construction schedule was included as a part of the contract papers for such work. For example, in the case of the contract for the steelwork of the main bridge, the construction schedule provided definite dates at which the towers had to be erected, at which the cables were to be completed, at which the floor steel erection was to be started and completed, etc. In general, of course, the sub-division of the work into separate construction contracts for particular parts made unnecesary the inclusion of the construction program in the contracts. The usual requirements in the contract papers were only for the completion of some certain part or parts of the work by a date which would comply with the requirements of the construction program of the Port Authority for the commencement of subsequent operations. An example of this type of requirement was in the case of the contract for the paving and railings on the main bridge structure, which required that the sidewalks and the installation of electrical conduits should be completed by a date in advance of that required for completion of the entire work under the contract, in order to conform to the construction program for the installation of electrical equipment on the bridge.

The various divisions of the work involved in the construction of the bridge and its approaches are given in Table 1. Contract HRB-1 was for preliminary borings, and since it is not a construction contract, it is omitted from this summary.

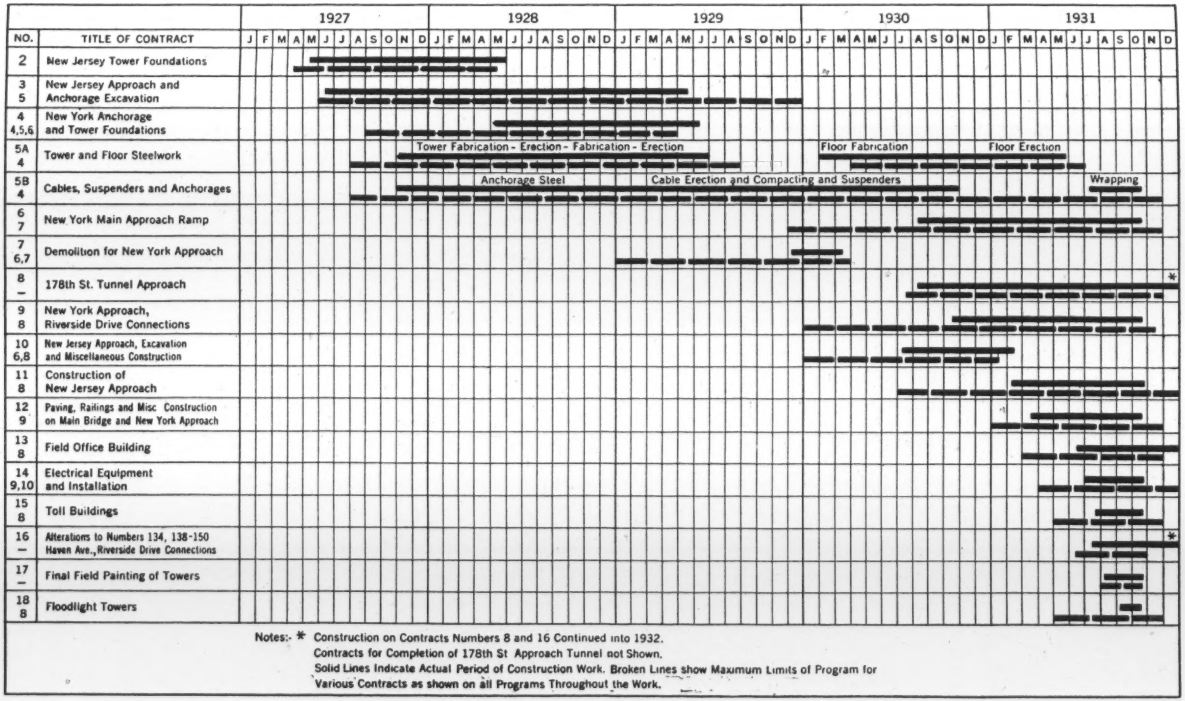


FIG. 4.—COMPOSITE CHART OF CONSTRUCTION PROGRAMS.

TABLE 1.—SUMMARY OF CONTRACTS FOR THE GEORGE WASHINGTON BRIDGE

Contract No. (1)	Description of work (2)	Dates of advertising work, opening bids, and awarding contracts, respectively (3)	Work completed (4)	Approximate cost (5)
HRB-2....	New Jersey Tower Foundation — two granite-faced piers, containing 36 000 cu. yd. of concrete and 1 400 cu. yd. of granite	Mar. 7, 1927 April 11, 1927 April 15, 1927	May 29, 1928	$1 059 000
HRB-3....	Excavation for New Jersey Anchorage and Approach — approximately 220 000 cu. yd. of trap-rock	May 12, 1927 May 31, 1927 June 2, 1927	May 25, 1929	1 150 000
HRB-4....	New York Anchorage and Tower Foundations — 9 700 cu. yd. of concrete in two tower bases; 107 000 cu. yd. of concrete for first part of anchorage	Feb. 9, 1928 Mar. 5, 1928 Mar. 8, 1928	Mar. 13, 1929	1 088 000
HRB-5A...	Towers and Floor Steel — Fabrication and Erection of Structural Steel — 40 000 tons for two towers and 17 000 tons for main bridge floor system	Aug. 4, 1927 Oct. 3, 1927 Oct. 13, 1927	Jan. 26, 1931	10 760 000
HRB-5B...	Cables, Suspenders and Anchorage Steelwork — manufacture and erection of four wire cables weighing 30 000 tons and 1 300 tons of 2¼-in. suspenders, fabrication of 5 000 tons of structural steel for the anchorages and erection of the steel in the New Jersey Anchorage	Aug. 4, 1927 Oct. 3, 1927 Oct. 13, 1927	Oct. 23, 1931	12 193 000
HRB-6....	Main Approach Ramp, New York — concrete arch over Riverside Drive, concrete and steel viaduct and miscellaneous construction in adjacent streets	June 19, 1930 July 14, 1930 July 17, 1930	Oct. 23, 1931	966 000
HRB-7....	Demolition and Removal of Buildings for the New York Approach — between 178th and 179th Streets, west of Fort Washington Avenue	Oct. 31, 1929 Nov. 18, 1929 Nov. 21, 1929	Feb. 27, 1930	150 000
HRB-8....	Vehicular Tunnel in West 178th Street of New York Approach — construction of surface and sub-surface plazas between Pinehurst and Fort Washington Avenues and tunnel structure (except finish, pavement, and lighting) from Fort Washington Avenue to Amsterdam Avenue	June 19, 1930 July 14, 1930 July 17, 1930	Nov. 1, 1931*	2 138 000
HRB-9....	Riverside Drive Connections of New York Approach — construction of roadways and reinforced concrete arch bridge	Aug. 21, 1930 Sept. 15, 1930 Sept. 18, 1930	Oct. 23, 1931	1 250 000
HRB-10...	New Jersey Approach Excavation and Miscellaneous Construction — excavation principally in rock and rough grading, and construction of walls and abutments	May 8, 1930 June 2, 1930 June 5, 1930	Jan. 8, 1931	323 000
HRB-11...	Paving and miscellaneous construction for the New Jersey Approach	Dec. 11, 1930 Jan. 5, 1931 Jan. 8, 1931	Oct. 27, 1931	565 000
HRB-12...	Paving, railings and miscellaneous construction on Main Bridge and New York Anchorage	Feb. 5, 1931 Mar. 2, 1931 Mar. 5, 1931	Oct. 23, 1931	493 000
HRB-13A..	Field Office Building in Fort Lee — a steel framed, masonry wall building for housing operation and maintenance forces and equipment	April 23, 1931 May 18, 1931 May 21, 1931	Dec. 30, 1931	195 000
HRB-13B..	Heating and ventilating equipment for Field Office Building	April 23, 1931 May 18, 1931 May 21, 1931	Dec. 20, 1931	13 000

* Essentially complete.

TABLE 1.—(*Continued*)

Contract No. (1)	Description of work (2)	Dates of advertising work, opening bids, and awarding contracts, respectively (3)	Work completed (4)	Approximate cost (5)
HRB–13*C*..	Electrical installation in Field Office Building	April 23, 1931 May 18, 1931 May 21, 1931	Jan. 8, 1932	$5 000
HRB–13*D*..	Plumbing system for Field Office Building..	April 23, 1931 May 18, 1931 May 21, 1931	Jan. 11, 1932	6 000
HRB–14...	Electrical Equipment and Installation on Bridge and Approaches — (except Riverside Drive connections)	May 7, 1931 May 25, 1931 May 28, 1931	Feb. 26, 1932	92 000
HRB–15...	Toll buildings.....	July 19, 1931† July 16, 1931 July 27, 1931	Oct. 23, 1931*	155 000
HRB–16...	Alterations to certain buildings encroaching on Riverside Drive Connections	June 4, 1931 June 22, 1931 June 25, 1931	Mar. 24, 1932	80 000
HRB–17...	Final field painting of towers.....	July 23, 1931† July 31, 1931 Aug. 1, 1931	Dec. 11, 1931	27 000
HRB–18...	Flood Light Towers, New Jersey Plaza — four steel towers mounting eight flood lights each	Aug. 20, 1931† Aug. 31, 1931 Sept. 10, 1931	Jan. 16, 1932	41 000
Total approximate cost of construction done under contracts				$32 752 000

*Essentially complete.
† Not advertised; contractors requested to bid.

Construction Procedure

All the construction contracts were let on the basis of competitive bids, the award being made in each case to the contractor who, in the judgment of the Port Authority, qualified as the lowest responsible bidder. Responsibility was determined by financial ability, experience, equipment, and organization to do the work. In most cases, the lowest bidder proved to be satisfactory, but occasionally it was deemed to be to the best interests of the Port Authority and the public to let the contract to one of the higher bidders.

The contracts were in two different types, which came to be known, respectively, as the "long" form and the "short" form. The short-form contract was developed from the long form, for use for comparatively simple work involving relatively small expenditures. Neither type, however, was set up with specific provisions or phraseology as "standard" to be used in every case, but each contract was examined critically, both by the Engineering and the Legal Departments of the Port Authority, in the light of the particular work to be performed, and of past experience and general policy, and modifications both in the substance and the wording were made from contract to contract.

The intent of the short-form contract was to minimize the routine in awarding the contract. The long-form contract provided a proposal that

was an offer to enter into the contract for the work; its acceptance created an agreement for the execution of the contract but not for the performance of the work. The short-form contract on the other hand was a letter form of proposal for performing the work and its acceptance effectuated the contract. Contracts in the short form were not publicly advertised; no cash deposit was required for the issuance of the contract papers; and no certified check was required with the proposal. The contract papers were sent to prospective bidders who were invited to submit proposals in duplicate on or before a definite date, generally without public opening. The successful bidder was required to furnish a surety bond before the return to him of one copy of the proposal with the Port Authority's acceptance endorsed thereon.

Except under unusual circumstances, contracts in the long form were advertised in engineering periodicals and in daily papers selected to give wide distribution throughout the Port District. In addition, letters calling attention to, and enclosing a copy of, the advertisement were sent to contractors likely to be interested in the particular work, and to agencies whose business it is to keep contractors advised of prospective work. The advertisement always described the work briefly, gave information for obtaining the contract papers and the deposit required for them, and stated the exact time and place for public opening of bids.

The proposals were opened promptly at the appointed hour, and no bids were received that were delivered after the opening had begun. As promptly as practicable after the opening, a conference was arranged for the lowest bidder with the Engineering Staff, and, except in cases where the lowest bidder was of known responsibility, conferences were also arranged with the second, third, and even the fourth low bidders. At each of these conferences, the work was thoroughly discussed and every effort was made to be sure that the contractor had acquainted himself thoroughly with the work and was satisfied with the sufficiency of his bid, as well as to determine his fitness to perform the work. Each prospective contractor, unless of well-known reputation, was required to furnish a number of references as to performance (which were later investigated by the Engineering Staff), and to furnish financial statements and banking and credit references, which were investigated by the Assistant Treasurer of the Port Authority.

Following the investigations, the Chief Engineer reported the findings, together with a recommendation as to award of the contract, to the General Manager of the Port Authority. The report was then transmitted to the Port Authority Commissioners who, by appropriate resolution, authorized the General Manager to execute the contract. As soon as such resolution became effective, the successful bidder was notified by the General Manager of the award and was furnished with duplicate copies of the contract forms, prepared for signature by the contractor. The return of these copies, properly executed, together with security for performance, was required within seven days. Upon their return, they were signed by the General Manager, thus effectuating the contract, and one copy was delivered to the contractor. This routine generally required three to four weeks from the date of the

receipt of bids, and therefore in some cases, where prompt performance was required, the procedure was varied, and the Commission authorized the General Manager, in advance, to make an award and execute the contract.

The procedure during performance of work under the contract was not materially different from the usual construction practice except as it was affected by the complexities of a large organization engaged, not in one great undertaking alone, but in several. The Chief Engineer was the responsible head, of course, but he could not keep personally in touch with all details under all contracts. Direct contact with the contractor was, as far as possible, through the Engineer of Construction, the Engineer of Design, and the Assistant Chief Engineer. The Chief Engineer, because of his heavy responsibilities and duties, necessarily placed wide discretionary responsibility upon the various Division heads. The contracts differentiated, by definition, between "Chief Engineer" and "Engineer," and the two words were used throughout with the purpose in view of reducing to a minimum the acts to be required by the Chief Engineer. The definitions provided that "Chief Engineer" meant the Chief Engineer (or in the event of his absence or disability the Assistant Chief Engineer) acting personally, and that "Engineer" meant the Chief Engineer acting either personally or through his duly authorized representatives.

All questions involving additional expenditures (extra work, adjustments of prices, changes in plans, delays, etc.) and all matters affecting the contractual relations were left to the decision of the Chief Engineer. Questions pertaining to performance of the work, such as quality of materials, workmanship, and orderly progress, were left to the authority of the Engineer. The Chief Engineer also (as is customary in contracts for public work) acted in the dual capacity of representative of the Port Authority, in charge of all operations, and as arbitrator in cases of dispute between the Engineer and the contractor.

Upon completion of the work under a contract, a detailed final inspection was made, usually by the Chief Engineer himself accompanied by one or more of the Division heads and by the Field Engineers directly in charge of the work. Usually, upon the completion of the work, there were a very small number of claims by the contractor for extra compensation, which had to be considered and adjusted. Furthermore, the total quantities involved in work under unit-price contracts were, of course, carefully checked, and agreement upon them was reached with the contractor. Following this procedure, the Chief Engineer issued a final certificate showing the entire amount of work performed and the compensation earned by the contractor.

In accordance with the requirements of the contract, the contractor was then required to furnish a detailed sworn statement of all outstanding liens, claims, and demands, just and unjust, of sub-contractors, material men, laborers, and third persons, and satisfactory evidence that the work was fully released from all such liens, claims, and demands. The contract provided that final payment to the contractor should act as a release to the Port Authority of all claims; and, as a matter of completing accounting records, each contractor was required to file a separate form of release in confirmation

of this contract provision, when final payment was made. At the time of preparing the final certificate, the Chief Engineer submitted to the General Manager a report upon completion of the contract, and, upon the transmittal of such report to the Commissioners, an appropriate resolution was adopted authorizing the final payment.

Contract and Specification Provisions

Considerable study was given to the preparation of the basic form of construction contract so that it would be clear and definite in its provisions and so that it would place full responsibility upon the contractor for doing his work and for damages resulting from his operations, but at the same time would protect the contractor as far as practicable against losses from unforeseen conditions. Some of the outstanding and unusual provisions will be discussed briefly in the order in which they occur in the contract papers.

Information for Bidders.—The Information for Bidders required each bidder to submit with his proposal a detailed list of plant and equipment, and a detailed description of the method and program of work he proposed to follow. This was not intended as something to which the contractor would be rigidly held, but rather as an indication to the Port Authority Staff of the fitness of the contractor and the thoroughness with which the problems had been studied. In this requirement, it was stated that the information would be regarded as confidential, but nevertheless it was sometimes found that considerable questioning was necessary before complete information could be elicited from the bidder. In spite of this reluctance of some bidders to disclose their plans, the writer believes that this provision is valuable to the determination of the bidder's responsibility.

Each bidder was required to deposit a certified check with his bid. The amount required for each contract was determined by judgment based on consideration of the work to be done, and not according to a fixed percentage of the estimated cost. A much higher percentage was required on the smaller contracts. The amount of the check ordinarily varied from 2% to 6% of the estimated cost of the work. The aim was to make it sufficiently substantial to afford a valuable indication of the contractor's financial ability; and investigation of the contractor's financial statements served to show whether the check was furnished by him or by a surety company.

On the greater part of the contracts, each bidder was required to submit an agreement with his proposal, signed by the surety that he proposed to have execute the performance bond. This agreement provided both that the surety would execute such a bond in case the contract was awarded to the bidder and also that, in case the bidder failed to execute the contract, the surety would pay to the Port Authority the liquidated damages set up in the Information for Bidders for such event. This "Agreement of Proposed Surety," and the certified check, provided the Port Authority with a double protection in case the low bidder should default and refuse to execute the contract.

Toward the end of the work, the Port Authority decided that this double protection was unnecessary and since, of the two forms, the deposit of the

certified check was considered the more valuable, the agreement of the proposed surety was abandoned. In all contracts, except the short form, the contractor was permitted, in lieu of the surety bond, to deposit acceptable security duly transferred or assigned to the Port Authority.

In unit-price contracts, the Information for Bidders contained a requirement that the successful bidder should furnish "an analysis of bid prices" when he returned the contract forms for signature by the Port Authority. This requirement proved helpful, and it is believed that it acts as a deterrent against the unbalancing of bid prices. The analysis was intended to be a complete "break down" of each unit price, to be used as a guide for the determination of an equitable price when, because of changes in character or quality of the work, it became necessary to make an adjustment. An analysis was not generally required in lump-sum contracts because it would be so complex as to be of little aid in any case of an adjustment of the lump-sum compensation.

Form of Contract.—A differentiation between the contract proper and the specifications was considered desirable, and accordingly the contract provisions proper were grouped in a section entitled "Form of Contract." All provisions to establish contractual relationships were included in this section and they could be changed only by the written consent of both parties. The specifications contained all provisions as to performance of work, materials, workmanship, and such conditions as were subject to change by the Chief Engineer, without modification of contractual relationships. The Form of Contract gave specific authorization to the Chief Engineer to modify the contract drawings and specifications, to require performance of work not shown on them, or to countermand directions for parts of the work.

Except in a few cases, unit-price contracts were used throughout because of their greater flexibility. The Port Authority contract, however, differs from the form commonly used in that it is not separable; that is, the prices paid under the separate items were not payments made for, or because of, work pertaining solely to those particular items, but were prices applied to the quantities of certain classes of work actually performed. When these prices were multiplied by the actual quantities involved in their respective items and the products were added, the total compensation to the contractor for the original contract work was obtained. The contractor was expected to include in the prices of these items—in any manner that he considered best—the cost of all work involved in the execution of the contract as a whole. The result of this provision in the unit-price contract was that it was unnecessary to provide a specific item for each class of work to be performed, and items were provided only for the principal classes of work, such as excavation, concrete, structural steel, and masonry, and for all classes of work, the amount of which was likely to vary from that originally estimated.

Extra work was defined in the Form of Contract as work required by the Engineer in addition to that required by the contract drawings and specifications which, in the judgment of the Chief Engineer, differed in general character from the work pertaining to the items under which unit prices were quoted. The Chief Engineer was authorized to make agreements with the

contractor for unit-price or lump-sum compensation for extra work, or to order the work performed on a basis of cost plus percentage. Compensation for cost plus percentage was the actual net cost in money to the contractor of materials and wages of applied labor (including premiums on Workmen's Compensation Insurance) plus 15%, and plus such rental for plant and equipment as the Chief Engineer deemed reasonable.

In order to insure fairness in compensating the contractor in the event that changes in plan or unforeseen conditions developed wide variation in quantities of work pertaining to any item, or involved changes in the character or quality of such work, the Form of Contract contained a provision giving the Chief Engineer authority to increase or decrease any of the unit prices quoted to such extent as, in his judgment, would effect the proper modification in the compensation warranted by such change in the work. In the case of such modifications, the Chief Engineer was also authorized to agree with the contractor upon lump-sum adjustment in lieu of adjustments of the unit prices. In the case of foundation work in particular, where the quantities may vary considerably from those estimated, this provision makes for fairness and equity because it provides for proper distribution of cost of plant and equipment, through adjustment in the unit prices.

The contracts did not provide any bonus in the event of completion before the specified date, but they did provide heavy liquidated damages for failure to complete within the time limits set up in the contract. In the handling of work of the magnitude of the George Washington Bridge under a program of construction—with numerous separate contracts for the various parts of the work—little, if any, advantage is obtained from bonuses for early completion. It is conceivable that even if full advantage could be taken of the time saved by early completion of some contracts, the aggregate of the bonuses paid therefor might be out of all proportion to the advantages gained.

It is essential to successful operation under a construction program that the various parts of the work be properly synchronized. While it is true that every opportunity must be taken to advance the completion of the entire work, nevertheless, a bonus on early completion of individual contracts might result in speeding up, out of all proportion, such parts of the work as could be easily expedited. This would result in certain parts being completed in advance of certain other parts just as essential, and no advantage would accrue from the bonus payment on the work completed ahead of schedule. For example, the contract for the cables was placed with one contractor and those for the two towers and the two anchorages with three other contractors. It is evident that any one of these three other contracts, under the stimulus of a bonus award, might have been completed far ahead of the others with no resultant advantage to the Port Authority, because the cable work could not be started prior to the completion of all of them.

Conversely, the beginning of the cable work would be delayed by the failure to complete on time any of the other three contracts. This latter condition was guarded against by liquidating the damages resultant therefrom in a very material sum. Such large sums of money were invested, and the prospective revenues from the bridges were so great that it was impossible

to liquidate fully the possible damages that might result by reason of delay, and, therefore, each contract was carefully considered by itself and the liquidated damages fixed in as large an amount as the contract would reasonably permit.

In many of the contracts, special provisions were made with regard to monthly payments to the contractor. It was expressly provided in the Form of Contract that the monthly payments were to be made as advances to the contractor to assist in financing the work. The sum retained out of each such monthly advance was regarded as security to the Port Authority for insuring performance of the work and protection against possible errors in the preparation of the monthly estimates. Inasmuch as the rate of interest on money borrowed by the Port Authority was generally less than the rate that would be paid by a contractor, the effort was made, in the interests of economy in the work as a whole, to keep the retained amount at the smallest value consistent with proper protection. This was accomplished in several ways. For example, in Contracts HRB-5*A* and HRB-5*B* (Table 1) provision was made that 10% of the monthly estimates would be retained at first, but that when the value of the work performed was equal to one-fourth the total contract price, only 7½% would be retained. When the value of the work reached one-half the total contract price, 5% would be retained; and when it reached three-fourths the total contract price, only 2½% would be retained. In other contracts, provision was made that 10% of the monthly estimate would be retained until the value of the work performed was one-half the total contract price, and thereafter the fixed sum of 5% of the total contract price would be retained. In contracts involving large sums of money the smaller percentages afford adequate security, and it is the writer's opinion that such provisions should be the usual practice in contracts for public work.

Unusual conditions affected the making of monthly payments on the contracts for the cables and for the towers and floor steel. The Port Authority could not profit by having a part of the work under these contracts completed far in advance of the date in the construction program (as, for example, one tower); nor by having the fabrication of the cable wire proceeding too rapidly in advance of the tower work; nor that of the floor system too far ahead of the completion of the cables. Since the amount of money invested in these parts of the work was so large and the interest charges on it correspondingly heavy, it was desirable to hold the contractors as closely as possible to the program established in the contract. This was accomplished by providing that the monthly estimates should in no case exceed a sum obtained by dividing the total contract price by the period of duration of the contract and multiplied by the total elapsed time, even if the work was performed with such rapidity that, in many cases, the value of the work accomplished was considerably in excess of the monthly estimates. This provision worked no injustice to the contractor, because, inasmuch as the limitation schedule was known in advance, its effect was fully discounted and the greater speed of construction was adopted because the savings in labor and use of plant were of such magnitude that they justified fully the increased financing burden placed upon the contractor.

As has been previously stated, full responsibility was placed upon the contractor for his work and for all damages and claims for damages resulting therefrom. The contractor was the insurer of the Port Authority against all contingencies arising out of the performance of the work; and he was required under the contract to warrant that he had financial ability, that he was experienced and competent, that he was familiar with rules and regulations affecting the work, that he had examined the contract drawings and specifications and the site, that the work could be performed satisfactorily, and that there was no collusion or fraud in connection with the contract. In short, the Port Authority placed full dependence upon the contractor to produce the finished work for the compensation named in the contract, and all costs and expenses involved in the work were his obligations. This course was justified because of the thorough studies, surveys, and sub-surface investigations which preceded the issuing of the contract documents, and because the complete and detailed drawings and specifications reduced to a minimum the uncertainties faced by the contractor.

The decisions of the Chief Engineer were, by the contract agreement, final on all questions relating to the work and its performance, and the contracts provided that the contractor must proceed with the work in all events. However, since the contractor was limited to money damage only, it provided that should he object to any decision of the Chief Engineer in regard to compensation, such disputes should be referred to arbitration under the rules of the Committee on Arbitration of the Chamber of Commerce of the State of New York. It was required that arbitrators should be engineers experienced in matters of the nature covered by the contract, selected from those listed as "Engineers" or "Consulting Engineers" in the Chamber of Commerce list of Official Arbitrators. They were empowered to make decisions as to questions of fact, which, in turn, could be submitted to appropriate Courts as a basis for the determination of legal rights and liabilities, including questions of the reasonableness of the Chief Engineer's decisions.

Wherever possible, latitude was given to bidders for applying their ingenuity, experience, and facilities toward developing the most economical methods for performing the work. The contract for the foundation piers for the New Jersey towers provided for competitive bidding on two types of construction; namely, pneumatic caissons or open coffer-dams. It had been found impossible to determine in advance which of these methods would be most economical. Therefore, complete drawings, specifications, and schedules of prices were provided because both methods and bidders were permitted to submit tenders on either or both methods, selection to be reserved until after the receipt of bids. The lowest bid for the coffer-dam method was considerably less than the lowest bid on the caisson method, and, therefore, the contract was let for the former. Provision was made, however, that if a bidder submitted tenders on both methods, the prices quoted on the one not adopted formed no part of the contract, so that, after the contract had been awarded, should the contractor decide to revert to the other and more expensive method, he would be obliged to perform the work under the prices quoted for the cheaper one. If, however, after the award of the contract it became advisable, in the

opinion of the Chief Engineer, to revert to the more expensive method, he had the authority to apply the unit prices of either method. This provision foreclosed the possibility of a contractor wilfully submitting a low bid on one method with the idea that, after he had been awarded the contract, he could revert to the other at a cost to the Port Authority that might possibly be higher than the low bid on that method.

For several reasons unusual contract provisions were necessary in connection with the steelwork of the main bridge (the cables, the towers, and the floor steel): (*a*) It was deemed advisable not to determine in advance of receiving bids whether all this work should be handled as one contract or whether the work should be separated into two contracts; and (*b*) it was thought desirable to call for bids on two alternate types of cables—parallel wire or eye-bar—and also on alternate methods of procedure and arrangement. By one method of procedure all four cables would be constructed simultaneously; by the alternate method one cable of each pair would be constructed first and the second subsequently while the floor steel was being erected. As for arrangement, bids could be made upon the basis of the two cables of each pair constructed side by side or one above the other.

Decision as to contracting for this work as a whole or in two parts could not be made in advance because it was felt that some contractors might wish to submit a more favorable tender for the entire work than they would if proposals were received separately for the cables and for the towers and floor steel, and these separate bids simply added together. In order to provide for these various alternate arrangements and types of cables, complete contract drawings and specifications were drawn, covering both types of cables, and six separate proposals were provided to cover the various conditions.

The contract for this entire work was designated "HRB-5" that for the towers and floor steel, "HRB-5*A*," and that for the cables, suspenders, and anchorage steelwork, "HRB-5*B*" (see Table 1). Contractors desiring to bid on both parts as a unit were instructed to make their bids on the proposals for Contract HRB-5; contractors desiring to bid on both parts of the structure separately were instructed to fill in the proposal blanks on Contracts HRB-5*A* and HRB-5*B*; and the contract provided that the acceptance of either one of these proposals would constitute a rejection of the other proposal. Finally, bidders desiring to bid on only one part of the work were instructed to fill in the proposal blank for Contract HRB-5*A* or Contract HRB-5*B*, as the case might be. In order to provide against the contingency of receiving a favorable quotation on one type of cable, and not receiving a suitable proposal for the towers for that type, contractors submitting proposals on Contract HRB-5*A* were required to bid on the designs for both the eye-bar and wire cables.

Due to the acute competition at the time between wire-cable interests and eye-bar cable interests, and the comparatively few contractors qualified to bid on this part of the work, there was the further possibility that contractors qualified to bid on the entire structure would do so without bidding separately on the towers and floor steel. This would result in a lack of competition on Contract HRB-5*A* for the towers and floor steel

as a separate unit. It was known that at least two or three concerns were interested in wire cables; but it was expected that they would not submit proposals for the towers and floor steel, and, therefore, it was possible that, because of failure to obtain a satisfactory proposal for the towers and floor steel alone, it might be impossible to take advantage of a low proposal on the cables. To prevent these contingencies, it was further required that, inasmuch as the submission of a proposal for the entire steelwork indicated a willingness on the part of the bidder to construct the towers and floor steel, either directly or through a sub-contractor, every bidder submitting a proposal for the entire steelwork was required also to submit a proposal for the towers and floor steel alone, or to arrange to have his proposed sub-contractor submit such a bid.

On October 3, 1927, proposals were received from five bidders, resulting in fourteen different combinations covering the steelwork. The lowest bids were accepted, decision being made thereby upon the various alternates, as follows:

(1) Wire cables were adopted in preference to eye-bar cables.
(2) The cables were to be placed in pairs, side by side, instead of one above the other.
(3) One cable of each pair was to be constructed first, so that erection of the floor steel could proceed at the earliest date, while the other two cables were being constructed.
(4) The steelwork was let to two different contractors, one for the cables, suspenders, and anchorage steelwork, and the other for the towers and floor steel.

Further studies made by the contractor for the cables subsequent to the letting of the contract developed a material advantage, both to the contractor and to the Port Authority, in the construction of all four cables simultaneously and, accordingly, although the contract had been awarded on the basis of successive erection of the cables, an agreement was made in the spring of 1929 whereby the cable contractor was to erect all four cables simultaneously under conditions such that the date for completion of the entire work, and incidentally that for completion of the entire structure, was materially advanced.

Specifications.—In the preparation of the specifications, the same care was given as in the preparation of the Form of Contract and the contract drawings, with the intent of insuring work of the highest quality in every respect. It was realized that the specifications were to serve as instructions to the bidder as to just what work was to be performed, and as to the standard by which the quality of work was to be controlled. To accomplish these purposes, it was realized that the specifications had to be complete, definite, and clear.

Each set of specifications was arranged in a series of chapters, so as to place the requirements before prospective bidders clearly and conveniently, and so as to provide for easy reference in the future by the contractor and the engineers. The particular chapter divisions adopted for any contract depended entirely upon the nature of the work to be performed under that contract, and no "standard" arrangement was considered desirable. An earnest effort was made continually not to fall into the error so frequently

made in the preparation of specifications, of blindly copying provisions made in a previous specification without carefully considering whether the provision applied properly or whether modifications were required to meet the particular conditions.

The practice was to have as the first section a chapter entitled "General Provisions." This was invariably begun with a carefully prepared statement of the work covered by the specifications. The statement was intended in no way as a limitation upon the work to be performed, since, by the terms of the contract, the contractor was obligated to do everything necessary or proper for, or incidental to, the completion of the particular part of the work covered by that contract. It was intended rather as a comprehensive picture of what was contemplated under the contract, so that the contractor could read the entire specifications with a better understanding. This statement was followed by the specific requirements of a general nature, such as the co-operation with contractors engaged in other parts of the construction work; precautions to be taken to safeguard against injury or damage to the work itself, to traffic, or to persons or property; temporary structures to be required for the work, such as fences, falsework, walkways, etc.; sanitary provisions to be required; and like general provisions.

The parts of the specifications that followed the "General Provisions" were subdivided into convenient chapters. In case the work involved many trades, as, for example, in a contract for a building, the chapters were separated into distinct classes of work which the general contractor might wish to sub-contract. If the work involved only one, or only a few types, of construction, however, the chapter divisions were based on the different classes of workmanship or construction involved. In a contract for fabrication and erection of structural steel, for example, separate chapters were devoted to design and detail, for the use of the drafting-room; to manufacture and workmanship, for convenience of the shop; to field work, for the requirements in erection; to materials, for the benefit of the mills; and, finally, to inspection and tests. As far as they were applicable, reference was made to the Standard Specifications of the American Society for Testing Materials, and, with the consent of that Society, reprints of the specifications referred to were bound with the printed contract papers, except where the reference was of only secondary importance.

Special care was given to avoid repetition, and to use phraseology that was plain, simple, straightforward, definite, and free from ambiguity. It was realized that the specifications, even more than the other parts of the contract, are read by different types of people—the engineer, the contractor, the material men, and, when disputes arise, the lawyers and the Courts—and that carefully chosen wording in the specifications, therefore, was essential. In general, because of the fact that the contractor was held responsible, under the terms of the contract, for all damages arising out of the work, the effort was made to specify the results desired, and not to specify methods by which the work should be performed, in order to avoid any implication, in case of damages, that the contractor was merely following instructions and that, therefore, the engineer shared the responsibility with him.

It was further borne in mind that repetition can easily lead to ambiguity and inconsistency and that, under such conditions, the Courts invariably rule that the contractor is entitled to the interpretation most favorable to his interests. In certain cases, it did become absolutely necessary to repeat, but in all such cases care was exercised to see that the two statements were consistent.

Throughout all the work, the Port Authority consistently refrained from specifying a proprietary article of a trade brand or of a particular manufacture. It sometimes happened, however, that certain materials were wanted the manufacture of which had become so standardized that it was literally impossible, except by going into disproportionately great detail and length, to indicate what was desired. In such cases, the article of specific brand or manufacture was clearly specified as a standard by which competing materials of the same nature were measured, and the good old time-worn phrase, "or an approved equal," was inserted.

Contractors

The successful bidders on work listed in Table 1 are: Contract HRB-2, Silas B. Mason, Incorporated; Contract HRB-3, Foley Brothers, Incorporated; Contract HRB-4, Arthur McMullen Company; Contract HRB-5*A*, McClintic-Marshall Company; Contract HRB-5*B*, John A. Roebling's Sons Company; Contracts HRB-6 and HRB-8, Cornell Contracting Corporation; Contract HRB-7, Klosk Contracting Company; Contract HRB-9, William P. McGarry Company; Contracts HRB-10 and HRB-11, George M. Brewster and Son, Incorporated; Contract HRB-12, Corbetta Concrete Corporation; Contract HRB-13*A*, Robert J. Murphy, Incorporated; Contract HRB-13*B*, Andrew I. Green, Incorporated; Contract HRB-13*C*, Hoffman-Elias, Incorporated; Contract HRB-13*D*, John Boyd Plumbing and Heating Company; Contract HRB-14, Beach Electric Company, Incorporated; Contract HRB-15, De Riso Construction Company; Contract HRB-16, Skolnick Building Corporation; Contract HRB-17, Salkind Company, Incorporated; and, Contract HRB-18, Auf der Heide Contracting Company.

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GEORGE WASHINGTON BRIDGE: DESIGN OF SUPERSTRUCTURE

BY ALLSTON DANA,[1] AND AKSEL ANDERSEN,[2] MEMBERS, AM. SOC. C. E., AND GEORGE M. RAPP,[3] ASSOC. M. AM. SOC. C. E.

SYNOPSIS

This paper gives a general description of the superstructure of the George Washington Bridge and an excerpt of the design specifications which contain certain unusual provisions. It covers in particular a detailed description of the design of the cables and floor system. It also records the weight of the structure as built to date (1932) and of the future additions as planned, and the principal quantities of the materials in the main bridge.

GENERAL DESCRIPTION

The main structure of the George Washington Bridge is of the suspension type, with a center span of 3 500 ft. between centers of towers, and suspended side spans of 650 and 610 ft. from center of tower to turning point at the anchorage on the New York and New Jersey sides, respectively. It is designed to carry a 90-ft. roadway flanked by two 10-ft. sidewalks on an upper deck and four electric railway tracks on a lower deck.[4] Only the upper deck has been built to date (1932) and the center portion is unpaved so that the bridge provides two 28-ft. 9-in. roadways and two sidewalks. The clear height under the completed bridge will vary from 195 ft. at the New York pierhead line to 210 ft. at the New Jersey pierhead line. Fig. 1 is a general view of the structure, and Fig. 2 shows the general plan and elevation.

The cables, towers, and anchorages are designed for the final loads, which are, as follows: Equivalent uniform dead load in center span, 39 000 lb. per ft.

[1] Engr. of Design, The Port of New York Authority, New York, N. Y.
[2] Asst. Engr. of Design, The Port of New York Authority, New York, N. Y.
[3] Asst. Engr., The Port of New York Authority, New York, N. Y.
[4] See Fig. 11, p. 25, and Fig. 20, p. 40.

of bridge; in side spans, 40 000 lb. per ft. of bridge; and live load in all spans, 8 000 lb. per ft. of bridge.

The assumed temperature variation from a normal of 50° was ± 55 degrees. The wind load was taken as 600 lb. per ft. on each deck and 300 lb. per ft. on the cables.

The cables, which are made up of parallel steel wires, are 3 ft. in diameter, and are arranged in pairs on either side of the roadway, with a distance of 106 ft. between centers of pairs, and a distance of 9 ft. between centers of cables in each pair. They have a center-span sag of 325 ft. under final dead load, and are about 15 ft. above the roadway at mid-span.

The floor structure is hung from the cables at each panel point by means of wire rope suspenders. The floor is cambered with its high point at mid-span, the profile in the center span consisting of two parabolic curves to which the grades of the side spans are tangent at the towers. The New York side span is on a 2.2% grade, while the New Jersey side span is on a 0.4% grade, the roadway being at a higher elevation at the New Jersey end than at the New York end, which takes up in part the difference in the height of land at the two ends of the bridge. The tops of both towers, however, are at the same elevation and the side-span cables slope from the towers at the same angle, but the turning point is set higher at the New Jersey anchorage, which accounts for the difference in side-span lengths.

The cables are supported in saddles on the top of two steel towers at an elevation of 591 ft. above mean high water. Each tower consists of two legs, separated sufficiently to allow the roadway to pass between them, and braced together at the top and just below the floor. Each leg is composed of an inner and an outer row of four columns, one pair of cables being supported over the inner row of columns of each leg. A full description of the design of the towers is given in another paper of this series.

At the turning points in the anchorages the cables are supported in saddles bearing on concrete buttresses and are deflected downward, the individual strands flaring apart in a distance of 90 ft. to connections with eye-bar anchor chains. These chains are embedded in concrete for a distance of 112 ft. in the New York anchorage and 150 ft. in the New Jersey anchorage and connect to anchor girders at their lower ends.

The New York anchorage is a U-shaped concrete structure consisting of a large block, in which the chains and girders are embedded, connected by low ribs to two forward buttresses supporting the cable saddles. In its completed state the anchorage is designed to have a granite facing that will give it the appearance of a single massive block of masonry, 300 ft. long, 190 ft. wide, and 125 ft. high. The weight of the completed anchorage will be about 520 000 000 lb. The resultant of the maximum cable pull of 248 000 000 lb. and the anchorage weight will have a slope of 1.85 on 1, and the maximum rock pressure will be about 12 tons per sq. ft. On the New Jersey side the anchorage chains and girders are set in rock tunnels filled with concrete.

The anchorages, towers, and cables form the main carrying system of the bridge, with the cables in each span taking the shape of an equilibrium polygon for the loads in that span. For normal temperature and no live

FIG. 1.—VIEW OF GEORGE WASHINGTON BRIDGE, 179TH STREET, NEW YORK, N. Y., TO FORT LEE, N. J.

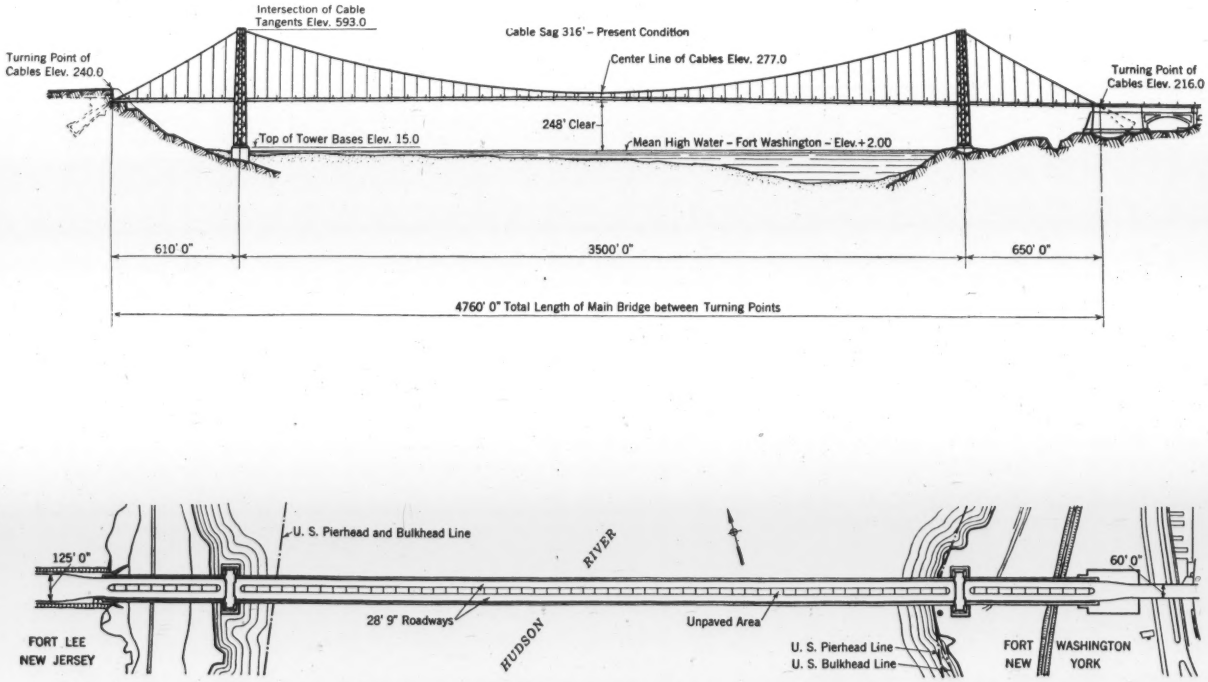


FIG. 2.—GENERAL PLAN AND ELEVATION OF BRIDGE PROPER.

load, the ratios of the sags in the different spans are such that the horizontal components of cable stress are equal for all three spans and the towers receive only vertical reactions.

Any increase of load in one span without a corresponding increase in the other spans would result in an unbalanced condition of horizontal forces at the top of the tower if the relative cable sags did not change in accordance with the change in load. Since friction prevents the cables from making the necessary adjustment of sags by slipping through the tower saddles, the saddles themselves must move horizontally, thereby increasing the sag in one span and decreasing it in the other, until equilibrium is established. Furthermore, changes in length of the side-span cables and anchorage steel, and even of the tower, from temperature or stress variations must be compensated by horizontal movement of the saddles, since otherwise the change in sag in the side span would be proportionally much greater than in the center span, with a resulting unbalancing of horizontal forces.

Erection of the suspended structure increases the unit stress in the cables and results in a riverward movement of the tower saddles. In order that, under final dead load, the points of intersection of the cable curves would be on the center lines of the towers, the saddles were placed on rollers 23 in. shoreward from their final positions prior to the construction of the cables.

With the bridge in operation the saddles are blocked against the tower tops so that the rollers do not function. Any movement of the saddles, therefore, forces the tower tops to move with them causing a horizontal reaction equal to the resistance of the tower to such movement. This horizontal reaction taken by the towers slightly dampens the movement of the saddles, since the horizontal components of the center and side spans are no longer exactly equal. However, the towers are so flexible, relatively, that the effect of their stiffness on the movement of the saddles is small. For the completed bridge the tops of the steel towers will have a total range of movement of 13.3 in. under the extreme conditions of live load and temperature.

A long-span flexible suspension bridge is unique among bridge structures in that the calculation of its stresses and resulting deformations cannot be based upon its geometrical dimensions before deformation without introducing a considerable error. The effect of the increased sag from live load is to reduce cable stress, including that due to load already on the bridge. The method used to determine the deformations for various load cases was a "cut-and-try" process of assuming the change in sag of the center span and then computing the stress deformation of the different members of the elastic system on the basis of the changed sag. It was then a matter of geometry to calculate the change in center-span sag resulting from the various stress, temperature, and cable polygon deformations. Only a few trials were necessary to obtain a sufficiently close check between the assumed and computed change in center-span sag.

At present, with only the upper deck in place, the bridge has no stiffening trusses. The dead load resists the live load deformation of the cable polygon, and, because of its great magnitude, offers ample stiffening effect. When the lower deck is built the bridge will have two stiffening trusses, 29 ft. deep

and 106 ft. apart. The upper chords of the stiffening trusses are now in place and form the chords of the wind trusses.

Subsequently in this paper detailed descriptions are given of the design of the cables and the floor system as built. Some of the features described were incorporated in the contract drawings on which bids were taken, while others were developed after the contract was awarded and the type of cables and method of erection were determined. No attempt is made in these descriptions to designate at what stage in the design any particular detail was established.

Design Specifications

Most specifications for the design of bridges in general, are scarcely applicable to long-span bridges. In fact, practically every long-span bridge has been designed in accordance with a special set of specifications intended to apply to that bridge alone. It was considered desirable to have a set of specifications, flexible and broad enough to be used for all bridges that the Port Authority was then planning and might design in the future. Accordingly, such specifications were drawn up, and have been applied, with slight modifications, to slabs, beams, and arches of concrete, as well as to long-span cantilever, arch, and suspension bridges.

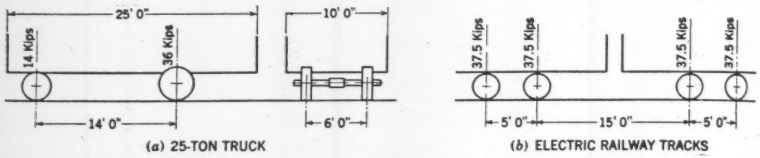
Live Load.—A novel feature of the Port Authority specifications is the treatment of the live load. It was realized that the intensity of live load, for which any part of a bridge structure should be designed, decreases as the extent of the area required to be loaded for maximum stress increases. This is true because the probable average congestion and percentage of heavier vehicles decreases as the loaded area increases. Furthermore, those parts of the structure, which require a large loaded area for maximum stress, have a higher percentage of dead load stress and are, therefore, better able to carry a live overload. Therefore, a uniform live load is specified which has a variable intensity, depending upon the loaded length and number of loaded lanes.

The basic uniform loads are: 100 lb. per sq. ft. of sidewalks, 250 lb. per sq. ft. of roadways, and 6 000 lb. per ft. of electric railway tracks. These basic loads are to be multiplied by two factors, K and C, which are functions, respectively, of the loaded length and the number of loaded lanes. Factor K is equal to $0.2 + \frac{160}{200 + l}$, in which, l is the loaded length, in feet; and C is equal to $0.5 + \frac{2}{n + 3}$, in which, n is the number of lanes loaded, considering a 10-ft. width of roadway as one lane. Where the live load is made up of lanes of different load intensities, the number of loaded lanes is to be obtained by dividing the total load per foot by the load per foot of the heaviest loaded lane; a minimum value of K of 0.25 corresponding to a loaded length of 3 000 ft., and a minimum value of C of 0.682 corresponding to eight lanes loaded, are specified. Thus, the minimum value of $K \times C$ is 0.170.

The uniform load is to be used alone, and not in combination with concentrated loads. However, since it is scarcely feasible to use uniform loads

in the design of members receiving their maximum stress from very short loads, concentrated loads are also specified. These loads are to be used as alternatives to the uniform loads, depending on which gives the larger stress. Fig. 3 shows the axle concentrations. Not more than one truck on any one lane, or the four axles on any one track, are to be considered. However, any number of lanes may be loaded, providing the concentrations are reduced by multiplying by the factor, C, which is the same as for uniform loads.

The ratio of impact to live load is, $\frac{150}{200+l} \times \frac{4}{3+n}$, in which, n and l are the same as in the live load formulas; l is to be taken as zero for concentrated loads. No impact is used for loaded lengths of more than 1 500 ft.



(a) 25-TON TRUCK (b) ELECTRIC RAILWAY TRACKS

FIG. 3.—AXLE LOADS.

Applying the specifications to the main carrying system of the George Washington Bridge, and using the width of roadway and sidewalks assumed at the time the design was made, the following live load, unreduced for loaded length and number of lanes, is obtained:

80-ft. roadway @	250 lb. per sq. ft.	=	20 000 lb. per ft.
Four electric railway tracks @	6 000 lb. per ft.	=	24 000 lb. per ft.
Two 10-ft. sidewalks @	100 lb. per sq. ft.	=	2 000 lb. per ft.
Total		=	46 000 lb. per ft.

As the loaded length is more than 3 000 ft. and the number of loaded lanes is eight, this load was multiplied by the minimum values of K and C, giving $46\,000 \times 0.170 = 7\,820$ lb. per ft., which was rounded up to 8 000 lb. per ft.

For maximum bending of the towers, certain spans should be unloaded. It was considered too severe an assumption to place maximum live load on some spans and no live load on other spans. It was assumed, therefore, that, for any loading case, either no span would be loaded with less than one-half the design live load, or no span would be loaded with more than one-half live load.

Most of the stiffening truss members receive their maximum stress from a comparatively short load. Since the stiffening truss is not a part of the main carrying system of the George Washington Bridge, an unreduced partial live load of 23 000 lb. per ft. of bridge, or one-half that given for the main carrying system, to be reduced by the factors, K and C, was specified and also an extended load of 4 000 lb. per ft., or one-half the reduced load given for the main carrying system placed over the entire bridge.

Wind Load.—The following wind pressures are specified to act as continuous, uniformly distributed loads of any length or position to produce maximum stress: A wind pressure of 300 lb. per lin. ft. along each deck, acting 6 ft. above the top of rail or roadway surface and at any track or lane of roadway, and a wind pressure of 30 lb. per sq. ft. of area of exposed structure. The leeward truss is considered fully exposed except where covered by solid floor structures or trains and vehicles. The wind loads used in the design of the George Washington Bridge of 600 lb. per ft. on each deck and 300 lb. per ft. on the cables are round figures arrived at from the specifications. The resulting wind loads are assumed to act either at right angles to the axis of the bridge, or at an angle of 45°, with transverse and longitudinal components each equal to two-thirds of these wind loads.

Lateral Force.—The specified lateral force for bridges carrying electric railway tracks is 10% of the live load on any two tracks, assumed to act 6 ft. above the top of rail.

Longitudinal Force.—The specified longitudinal force from breaking or traction is 20% of one 25-ton truck on each of any two lanes of the roadway and 20% of the specified uniform live load, not exceeding 1 000 ft. in length, on any two tracks. The forces from the two trucks or trains may act in the same or in opposite directions. This longitudinal force was found to be 720 000 lb. for the center span of the George Washington Bridge.

Permissible Unit Stresses.—The following tabulation gives the permissible unit stresses, in pounds per square inch:

Tension, Net Section:		
Rolled carbon steel	20 000	
Rolled silicon steel	27 000	
Rolled nickel steel	33 000	
Heat-treated eye-bars, low strength	33 000	For total stress, inclusive of secondary stress
Heat-treated eye-bars, high strength	50 000	For total stress, inclusive of secondary stress
Reinforcing steel	18 000	
Cold-drawn cable wire	82 000	For total stress, exclusive of secondary stress
Compression, Gross Section (in which, l = unsupported length of member, and r = least radius of gyration):		
Rolled carbon steel	$20\,000 - 60\frac{l}{r}$, maximum, 17 000	
Rolled silicon steel	$27\,000 - 80\frac{l}{r}$, maximum, 23 000	
Rolled nickel steel	$33\,000 - 100\frac{l}{r}$, maximum, 28 000	
Steel castings	20 000	
Iron castings	12 000	
Bending on Extreme Fibers:		
Riveted Girders and Rolled Beams:		
Tension, Net Section:		
Carbon steel	20 000	
Silicon steel	27 000	
Nickel steel	33 000	

Compression, Gross Section (in which, l = unsupported length of flange, and b = width of flange):

Carbon steel	$20\,000 - 200\frac{l}{b}$, maximum, 17 000
Silicon steel	$27\,000 - 270\frac{l}{b}$, maximum, 23 000
Nickel steel	$33\,000 - 330\frac{l}{b}$, maximum, 28 000
Forged carbon steel	18 000
Steel castings	15 000
Pins, carbon steel	30 000
Pins, heat-treated	50 000

Shear:

Girder Webs, Gross Section:

Carbon steel	12 500
Silicon steel	17 000
Nickel steel	20 000
Carbon-steel pins	15 000
Heat-treated pins	25 000
Power-driven rivets and turned bolts	15 000
Hand-driven rivets and unfinished bolts	10 000
Steel castings	12 000
Iron castings	5 000

Bearing:

Carbon steel, cast steel, and carbon-steel pins	30 000
Silicon steel	40 000
Nickel steel	50 000
Heat-treated pins	55 000
Power-driven rivets and turned bolts	30 000
Hand-driven rivets and unfinished bolts	20 000
Rollers, per linear inch, (in which, d = diameter of roller, in inches):	
Ordinary carbon steel	$800d$
Steel, minimum elastic limit, 60 000	$1\,200d$

All the permissible unit stresses given in the foregoing tabulation are taken from the general Port Authority specifications and were used for the George Washington Bridge with the exception that the following values were used for power-driven rivets and turned bolts: Shear, 12 500 lb. per sq. in.; and bearing, 25 000 lb. per sq. in.

The following combinations of stresses are specified for the proportioning of any member:

1.—Dead load, live load, impact, and temperature;
2.—Dead load combined with wind, or longitudinal force, or lateral force.
3.—Lateral force combined with wind, or longitudinal force;
4.—Longitudinal force combined with wind;
5.—Dead load, live load, impact, and temperature combined with either wind load of one-half the intensity of that specified under "Wind Load," or longitudinal force, or lateral force.
6.—Dead load, one-half live load, one-half impact, and temperature combined with wind.

For Combinations 1, 2, 3, and 4, the permissible basic unit stresses as given are to be used. For Combinations 5 and 6, unit stresses 10% higher than the basic unit stresses may be used. The permissible unit stresses for the combination of primary stresses and secondary stresses are 25% greater than those specified for primary stresses alone.

The materials for which the foregoing permissible unit stresses are given, are required to have the following minimum yield points and ultimate strengths, in pounds per square inch:

	Minimum yield point.	Minimum ultimate strength.
Rolled carbon steel	35 000	58 000
Rolled silicon steel	45 000	80 000
Rolled nickel steel	55 000	90 000
Rivet steel	30 000	52 000
Pins and rollers	35 000	65 000
Heat-treated pins, bolts, and rollers	60 000	105 000
Forged carbon steel	35 000	70 000
Cast steel	35 000	65 000
Heat-treated eye-bars, low strength	50 000	80 000
Heat-treated eye-bars, high strength	75 000	105 000
Cable wire	150 000	220 000
Reinforcing steel	33 000	55 000

Weight of Structure and Principal Quantities for Main Bridge

Dead Load.—The make-up of the dead loads for the center span is, as follows:

	Dead loads, in pounds per foot:	
Present Structure:		
Cables, including wrapping and hand ropes	11 120	
Suspenders, including cable bands and other details	620	
Top chords	830	
Wind diagonals	185	
Main floor-beams	2 080	
Stringers and bracing	2 085	
Secondary floor-beams	835	
Bulb beams and tie-rods	1 075	
Sidewalk steel, including fascia girders	530	
Curbs	495	
Railings	260	
Side roadway slabs (two @ 28 ft. 9 in.)	4 850	
Sidewalk slabs	950	
Electrical equipment	120	
Total for present structure		26 035
Additions for Completion of Upper Deck:		
Alterations to inside curbs and railings	—75	
Center roadway slab (30 ft. 6 in.)	2 610	
Total additions for completion of upper deck.		2 535
Total for completed upper deck		28 570

Total for completed upper deck (*Brought forward*)		28 570
Lower Deck:		
Bottom chords, diagonals, and verticals	1 570	
Main floor-beams	1 200	
Longitudinal girders	1 210	
Lateral bracing	100	
Intermediate floor-beams	500	
Stringers and bracing	620	
Braking trusses	60	
Rails, ties, fastenings, etc	2 100	
Total for lower deck		7 360
Total for completed bridge		35 930
Allowance for possible additions		3 070
Total design load: Center span		39 000

The weights of cables, suspenders, and wind diagonals are not uniform; the loads given for them in the foregoing tabulation are equivalent uniform loads which produce the same moment at the middle of the span as their actual weights.

The corresponding dead load for the side spans is 40 000 lb. per ft., the greater weight being due to the steeper slope of the cables, the longer suspenders, and the heavier cable bands. For calculating the dead load cable polygon (which was required for determining the length of suspenders and indirectly for setting the strands to the proper sag during the erection of the cable), the actual panel load concentrations were used instead of uniform loads.

Principal Quantities for Bridge Proper.—As built to date (1932), the main structure of the George Washington Bridge, exclusive of the floor in the anchorages, contains the following quantities of steel, in pounds:

Cable wire and suspender rope		60 000 000
Silicon Steel:		
Towers	47 200 000	
Floor	16 200 000	
Total		63 400 000
Carbon Steel:		
Towers	37 000 000	
Floor	22 400 000	
Anchorages	4 400 000	
Total		63 800 000
Heat-treated eye-bars, pins, and bolts		9 200 000
Cast steel		5 300 000
Railings, light standards, conduits, etc		1 800 000
Total steel in present structure		203 500 000
The estimated quantity of steel, in pounds, in the future lower deck, is		25 000 000

The quantities of concrete and granite facing, in the main structure, in cubic yards, are:

New York tower foundation	11 500
New Jersey tower foundation	38 000
New York anchorage	113 000
New Jersey anchorage	29 000
Roadway and sidewalk slabs on suspended structure	6 700
Total concrete and granite in present structure	198 200
Reinforcing rods, in pounds, in concrete of main structure	3 000 000
Estimated concrete, in cubic yards, to finish roadway pavement	3 000
Estimated granite and concrete, in cubic yards, to complete New York anchorage	27 000

Design of Cables, Anchorages, and Suspenders

Cables.—Each cable is composed of 26 474 galvanized steel wires, 0.196 in. in diameter (No. 6 B. W. G. (Birmingham wire gauge)), over galvanizing, laid parallel to form a compact cylinder 36 in. in diameter. There are 800 sq. in. of steel area in each cable, and the length along the cable from strand shoes to strand shoes is nearly 1 mile. The total weight of wire in the four cables is 56 600 000 lb., and the total length of single wire is about 105 000 miles, or enough to extend slightly more than four times around the earth at the equator.

The cable wire is cold-drawn steel having a specified minimum ultimate strength of 220 000 lb. per sq. in., and a minimum yield point of 150 000 lb. per sq. in. The actual wire furnished by the contractor showed somewhat higher strength, however, the ultimate strength averaging about 234 000 lb. per sq. in., and the yield point about 184 000 lb. per sq. in.

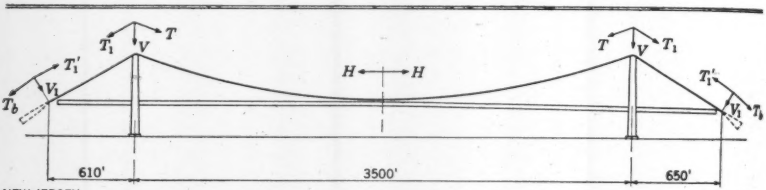
The cables are designed for a maximum unit stress of 82 000 lb. per sq. in., exclusive of secondary stresses. This maximum stress occurs in the side spans adjacent to the tower saddles. The maximum direct unit stresses in the cable at other points are in direct proportion to the secant of the angle of inclination of the cable with the horizontal: In the center span adjacent to the tower, it is 74 000 lb. per sq. in., or 10% less than the maximum; at the middle of the main span, it has a minimum value of 69 000 lb. per sq. in., or 16% less than the maximum; at the anchorage saddles and in the backstays adjacent to the eye-bars, it is 78 000 lb. per sq. in., or 5% less than the maximum. Table 1 shows the forces in the cables at the towers and anchorages.

On account of the relatively short, stiff side spans and the resulting small angular distortions of the cable, the maximum secondary stresses do not occur at the point of maximum direct stress, however, but rather on the opposite side of the saddle in the center span. To determine the intensity of the secondary stresses in the cable at this point field measurements of strain were made on 150 wires on the periphery of the cable during different stages of

construction when the cable distortions were large and varying. (See Table 2.) A study of these measurements and the measurements of angular deflection that were made simultaneously with them—when applied to the maximum direct stress and simultaneous angular deflection of the cable on each side of the tower saddle—indicates that the bending stresses at these points will be approximately 2 000 lb. per sq. in. on the side-span side and 5 000 lb. per sq. in. on the center-span side.

The determination of the maximum cable pull and the required sectional area is a comparatively simple matter. By taking moments about the low point in the cable (mid-span) and dividing by the sag of the cables, the horizontal component of the cable pull is found. Multiplying this by the secant of the maximum inclination of the cables with the horizontal gives

TABLE 1.—FORCES, IN MILLIONS OF POUNDS, FOR FOUR CABLES



	T_b	V_1	T_1'	T_1	V	T		H		T	V	T_1	T_1'	V_1	T_b
Dead	208	34	208	218	184	196		184		196	185	218	207	34	207
Live	38	7	38	40	36	36		34		36	36	41	39	7	39
Temp.	2	0	2	2	1	2		2		2	1	2	2	0	2
Total	248	41	248	260	221	234		220		234	222	261	248	41	248

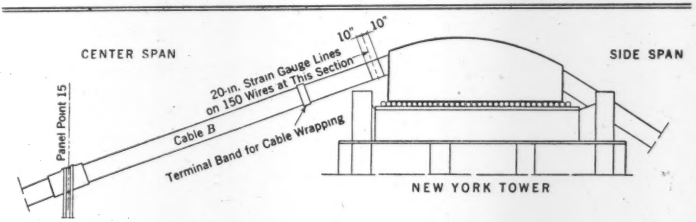
the maximum cable pull and then, dividing by the allowable unit stress, the required sectional area is found. However, the cables stretch under live load and change length with temperature, causing a change in sag of sufficient amount to be considered in the computation. By a process of "cut-and-try," or by means of a more complicated direct solution, the sag and the pull of the cables can be found for any loading case. For the case giving maximum cable pull, namely, live load on all spans and minimum temperature, the sag of the cables was found to be 2.0 ft. greater than normal, and the maximum pull in each cable, to be 65 300 000 lb. Dividing this by the allowable unit stress of 82 000 lb. per sq. in. gave a required area of 796 sq. in.

In proportioning the cable certain factors had to receive consideration: For convenience in erection and in anchoring, the cable is divided into component parts called strands, all strands having the same number of wires. The number of wires in each strand must be limited so that its weight is not too great for easy handling in the field during erection. It was decided that 61 strands of 434 wires, $4\frac{1}{2}$ in. in diameter—requiring lifts at the tower of about 110 tons and pulls at the anchorages of about 130 tons—would not impose unreasonable requirements in the design of the erection equipment, and would give fewer units to handle compared with the next

alternative of using 91 lighter strands. As considerable erection time is consumed in handling and adjusting the completed strands there is an appreciable saving resulting from the use of the 61-strand assembly. The use of 61 rather than 91 strands also required fewer anchorage eye-bars and strand shoes, which resulted in a simpler and more economical anchorage.

The 61 strands are laid up in the form of a hexagon during the construction of the cables and, later, are compacted to a circular shape throughout the three spans. Where they pass through the saddles on top of the towers

TABLE 2.—RESULTS OF CABLE-BENDING STRESS MEASUREMENTS

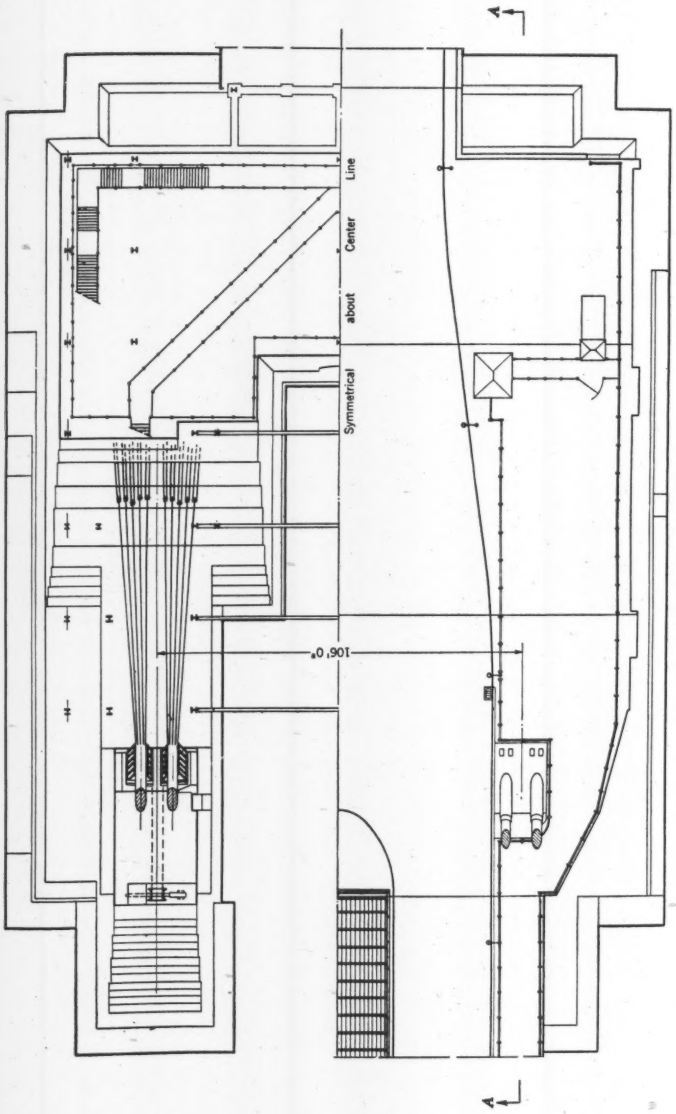


(a) LOCATION OF STRAIN MEASUREMENTS ON CABLE FOR DETERMINATION OF EFFECT OF BENDING

Erection Stage	Deflection at PP15 in Ft.		Average Stress in Kips per Square Inch		Stress Diagrams 0 5 10 15 20 (Kips per Square Inch)
	Measured	Calculated	Measured	Calculated	
N. J. PP43 N. Y. (b)	0	0	Starting Point	19.6	25 30
(c) 18 18	1.54	1.59	21.5	20.8	
(d) 23 22	1.97	1.89	25.6	22.7	
(e) 32 32	1.65	1.41	28.6	27.2	35 40 45
(f) All Steel Erected	0.37	0.38	32.8	32.9	
(g) Concrete Mixer 31	1.49	1.52	36.5	35.2	
(h) Bridge Completed	0.70	0.74	41.9	41.6	

and at the anchorages, however, the cables retain their original hexagonal shape, the saddles being designed for that condition.

In order to design the cable bands that support the suspenders, some knowledge had to be obtained beforehand of the approximate diameter of the compacted cables. This was determined by applying the formula, $D = k \times 1.05 \times d \sqrt{N}$, in which, D is the diameter of the cable, k is a constant, d is the diameter of the individual wires, and N is the number of wires. The constant, k, was determined from a study of the diameter

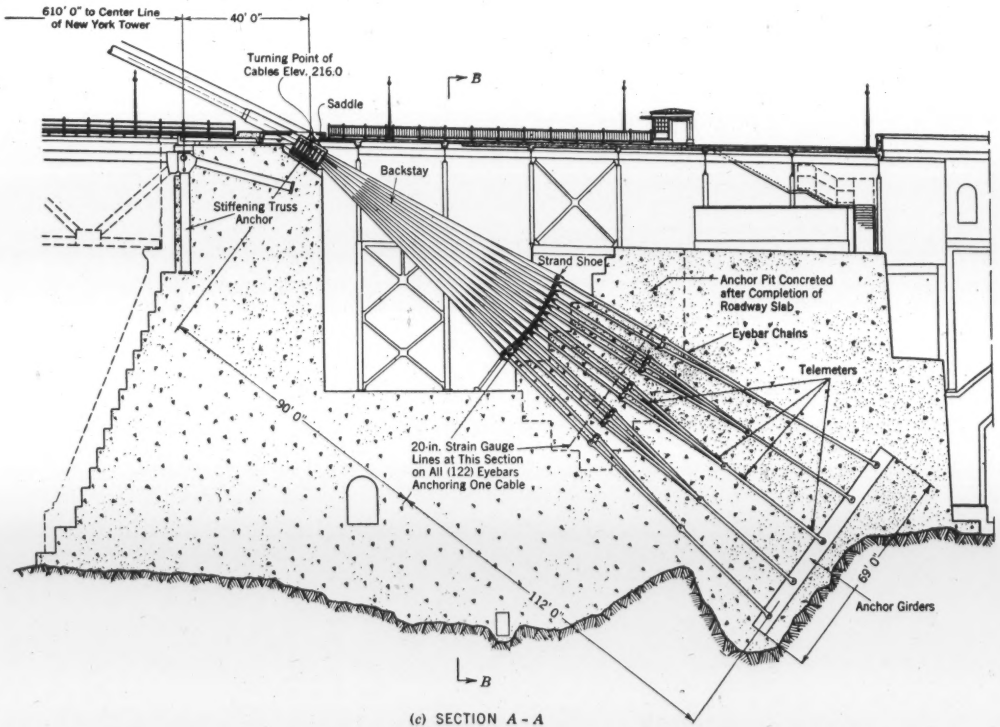


(a) HALF PLAN WITH ROADWAY REMOVED

(b) HALF PLAN OF ROADWAY LEVEL

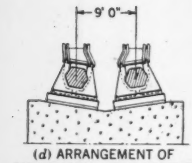
FIG. 4.—PLAN OF NEW YORK ANCHORAGE (SEE, ALSO, FIGS. 5 AND 6).

FIG. 4.—PLAN OF NEW YORK ANCHORAGE (SEE, ALSO, FIGS. 5 AND 6).

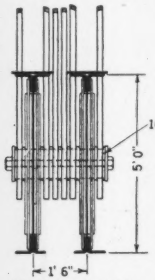


(c) SECTION *A - A*

FIG. 5.—LONGITUDINAL SECTION *A-A*, NEW YORK ANCHORAGE (SEE, ALSO, FIGS. 4 AND 6).

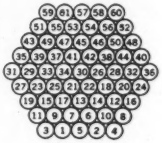


(*d*) ARRANGEMENT OF ANCHORAGE CABLE SADDLES

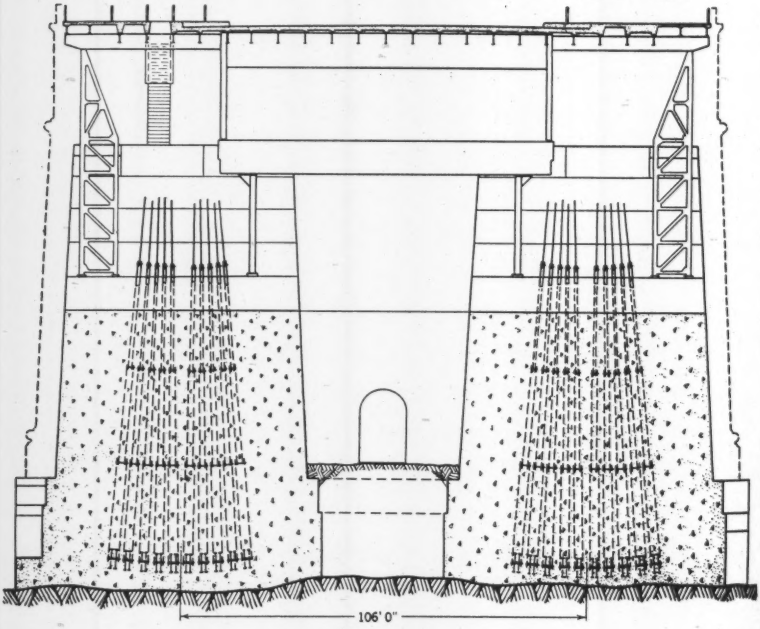


(*e*) SECTION THROUGH ANCHOR GIRDER

(*f*) ARRANGEMENT OF STRANDS IN UNCOMPACTED CABLE



(*g*) ARRANGEMENT OF STRAND SHOES



(*h*) SECTION *B* - *B*

FIG. 6.—TYPICAL SECTIONS, NEW YORK ANCHORAGE (SEE, ALSO, FIGS. 4 AND 5).

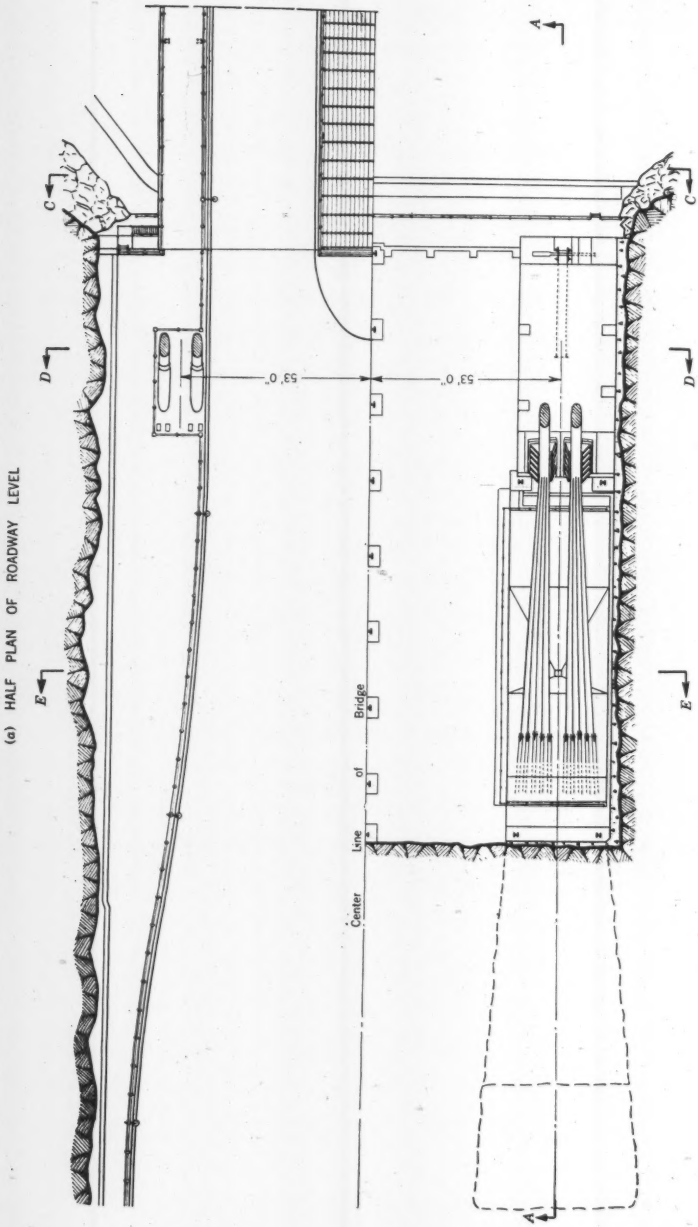
of the cables of several other large suspension bridges. Expressed in another way, there is a minimum of about 10% voids in a perfectly compacted group of wires, but as this ideal condition cannot be obtained, there is found by experience to be about 10 to 12% additional voids. On this basis, it was calculated that the diameter of each cable would be 36 in. Later, as a result of experimental compacting on a full-sized section of sample cable, constructed to determine the bore of the cable bands, it was found that the cable could be squeezed to a diameter of 35¼ in., which gives 21% of voids.

Anchorage Steelwork.—The maximum cable stress, cable section, and number of strands being determined, one of the next problems was to design the steel anchorages. The positions of the turning points or anchorage-cable saddles were determined so as to be below the main floor deck, and to give approximately the same slope to the cables in the two side spans. From these points the cables deflect downward to the anchorage steel through a vertical angle of 9° 28′. The two cables of each pair separate, each cable deflecting through lateral angles of 1° 34′ in New York and 1° 19′ in New Jersey, and the 61 strands of each cable flare or splay apart both vertically and laterally, each strand connecting at its end to an independent pair of eye-bars. (See Figs. 4 to 9.)

The vertical deflection angle of the cables was made the same at both anchorages in order to keep the details of the saddles the same; it was made sufficiently large, but no larger than necessary, to prevent the topmost strands from lifting off the saddle in the worse condition of upward deflection in the side span. The deflection angle of the top strands, therefore, is only about 1 degree.

The lateral deflection of the cables was necessary because the 9-ft. spacing of the cables (which was made the minimum consistent with the space required for spinning, compacting, and wrapping the cables, and for the design of the saddles), could not be maintained in the anchorages on account of the greater space required for the anchor steel. The cables, therefore, diverge in two straight lines from the 9-ft. spacing at the saddles to a 20-ft. spacing at the bottom of the anchorage steelwork, where the ten double girders are placed with the minimum lateral spacing of 4 ft., center to center. (See Figs. 6 and 9.)

In order to avoid the use of tension ties at the turning point between the cables to take the lateral thrust caused by these horizontal deflections, the anchorage saddles were rotated about the center line of the side-span cables at that point until they were in the oblique plane formed by this line and the center line of the splayed back-stay cable and anchorage steel. This placed them in planes making dihedral angles with the vertical planes of the side span cables equal to 9° 24′ and 7° 56′, respectively, on the New York and New Jersey sides. In this way the resultant of the pulls in the side span and back-stay cables is a simple thrust against the masonry of the concrete buttresses.



(a) HALF PLAN OF ROADWAY LEVEL

(b) HALF PLAN WITH ROADWAY REMOVED

FIG. 7.—PLAN OF NEW JERSEY ANCHORAGE (SEE, ALSO, FIGS. 8 AND 9).

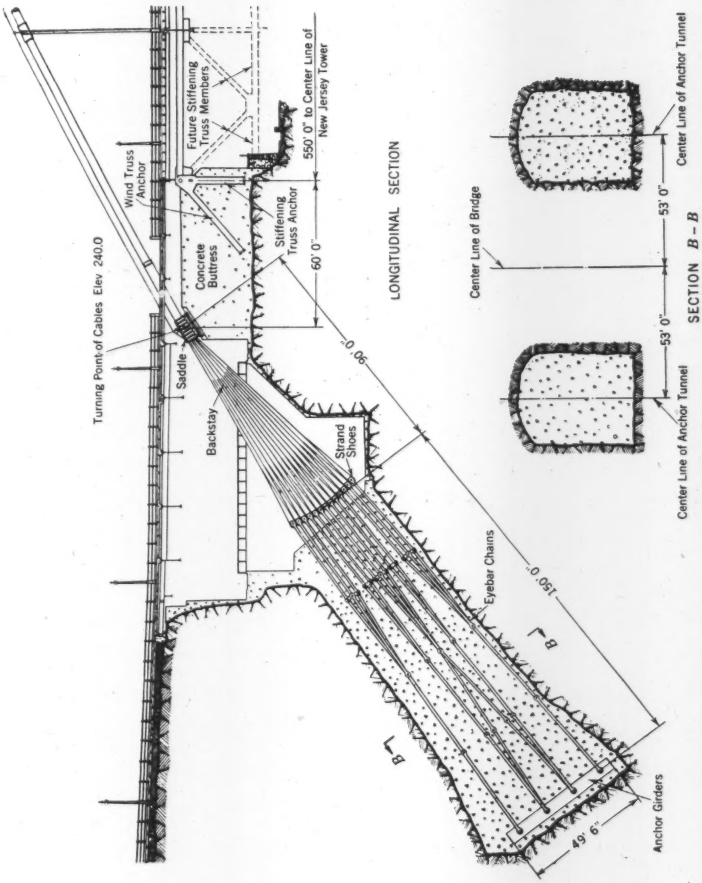
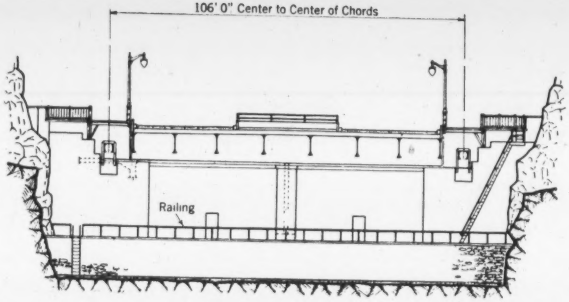
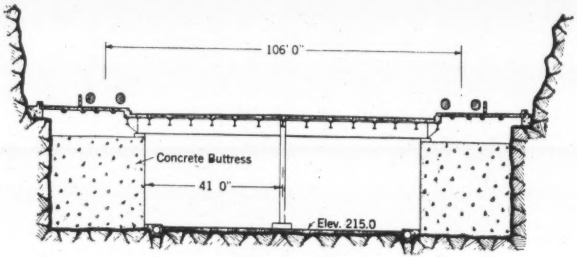


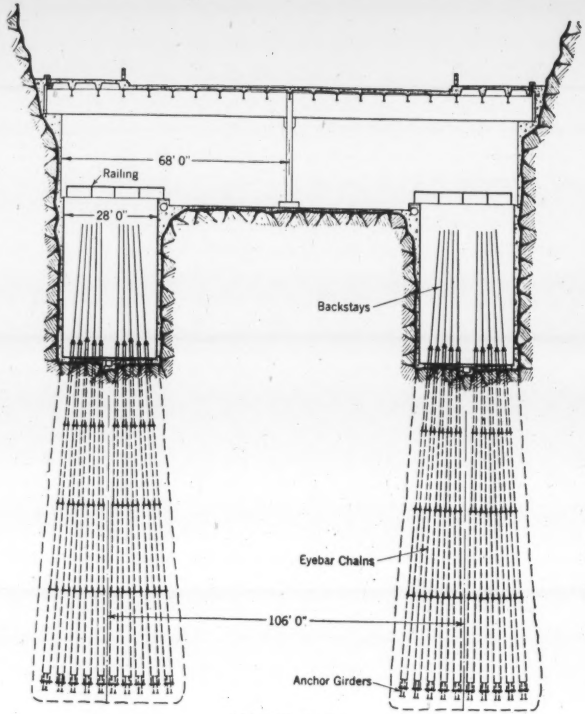
FIG. 8.—LONGITUDINAL SECTION A-4, NEW JERSEY ANCHORAGE (SEE, ALSO, FIGS. 7 AND 9).



END ELEVATION *C-C*



SECTION *D-D*



SECTION *E-E*

FIG. 9.—TYPICAL SECTIONS, NEW JERSEY ANCHORAGE (SEE, ALSO, FIGS. 7 AND 8).

Instead of supporting the cables in their hexagonal shape in the conventional grooved saddles, thence passing to a circular shape again and into separate splay collars from which the strands diverge to their anchors, the cables were splayed directly from the mouth of the saddles, and the saddle details were made to accommodate this. This solution was possible because the cables at this point are beneath and are protected by the floor and, therefore, did not require compacting and wrapping for protection against the weather. By eliminating separate splay collars the length of the cables was reduced and their construction simplified.

In the New York anchorage there are three tiers or lengths of eye-bars between the strand shoes and the anchor girders, whereas in the New Jersey anchorage there are four tiers, the distance between the shoes and the girders being 112 ft. in New York and 150 ft. in New Jersey. This greater length of the New Jersey anchors resulted from using a more conservative value for the shearing stress in rock than in concrete. Each vertical row of strand shoes is connected by a chain of eye-bars to a separate anchor girder, the thirteen lines of eye-bars in the upper tier of a typical chain converging in groups of two and three to form the five lines of eye-bars of the lower tiers. Each anchor girder is composed of two plate girders fastened together with diaphragms and batten-plates.

In the New York anchorage the lateral spacing of the strand shoes and that of the girders are in proportion to their distances from the saddle, so that the lateral flare of each eye-bar chain is a continuation of the lateral flare of the strands which it anchors; and the pins in the girders were located so that each line of eye-bars in a chain is a continuation of the group of strands connecting to it. This straight-line pull arrangement made it unnecessary to use lateral separators between the strand shoes, although vertical separators were necessary on account of the converging of the groups of upper tier eye-bars previously mentioned.

A similar straight-line pull arrangement was not practicable in the New Jersey anchorage. On account of the greater length of the New Jersey anchor chains such an arrangement would have required longer girders and a wider spacing between them, whereas it was desirable to keep the size of the rock tunnels, in which the anchorage steel was embedded, to a minimum. The lateral spacing, therefore, was made the same as in the New York anchorage and the girders were made considerably shorter, thus causing a change in direction at the shoes between the strands and the eye-bars both vertically and laterally. This required lateral as well as vertical separators between the strand shoes to take care of the thrusts. Furthermore, during the spinning of the cables, prior to the completion of the top strands, it required special holding down devices to resist the unbalanced upward thrust.

The maximum pull in each cable at the anchorages is 62 000 000 lb. This is assumed to be distributed equally among the 61 strands and pairs of eye-bars. Heat-treated eye-bars were used with a specified minimum yield point and ultimate strength of 50 000 and 75 000 lb. per sq. in., respectively, and

with an allowable unit stress of 30 000 lb. per sq. in., exclusive of secondary stress. A sectional area of 16.9 sq. in. was required for each bar, and in all but the upper tier 10 by $1\frac{3}{4}$-in. bars were used. In the upper tier, 10 by $1\frac{7}{8}$-in. bars were used, the heavier section being provided to take care of the small but unavoidable bending stresses to which these bars are subjected. As far as possible all bars in both anchorages were made 38 ft. 4 in. long, which is the maximum for full-sized tests.

The use of ordinary carbon-steel eye-bars instead of the heat-treated bars was considered, but the greater weight and increased packing dimensions would have more than offset the lower unit price.

The strand shoes and girders were set laterally on circular arcs corresponding to the lateral flare of the strands and eye-bar chains. The strand shoes, however, were set vertically on a sharper circular arc centering at a point midway between the shoes and the saddle, thus making the lengths of the flared strands inversely proportional to the secant of their angles of flare and in this way equalizing their stress.

Further precautions were taken to insure an equal distribution of stress in the back-stay strands by eliminating the sag due to the relatively heavy concentrations from the strand shoes, eye-bar heads, and pin details. During cable construction the top tier of bars cannot be concreted in, and the entire assembly, if unsupported, would sag at the expense of an increase of stress in the lower strands. Therefore, column supports were provided under each chain at the strand shoes, which held the strands in their correct final position during and after construction of the cables, and until the eye-bars were embedded in concrete.

Another source of inequality of stress in the back-stay strands is inherent in the successive adjustment of the strands from the bottom up as the cable is built. As each new strand is adjusted for side-span sag it is lowered into place in the anchorage saddle and, simultaneously, the shoe is pulled back to its final position. During the spinning of the strand the shoe is held in a position forward of its final position by a strand leg which is pinned to the anchorage eye-bars. The pulling back is done by a pulling jack and rope which connect to the strand leg and to the pin at the lower end of the upper tier of eye-bars, these bars being released from stress during the process. In order to have clearance for inserting the shims required for the adjustment, the shoe is pulled back a little farther than its final position and as the pulling jack is released the shoe moves forward the amount of this shim clearance and also the amount of the elastic stretch of the upper tier of eye-bars to which the pull is transferred from the jack. This slight forward motion of the shoe allows the strand to slip forward in the saddle, but only to a partial extent on account of the friction of the strand on the strands below in the saddle. Consequently, the stress in the strand is reduced by the amount of this friction, and the stress in the previously adjusted strands is increased.

This unbalancing of stress in the strands, and its amount, was discovered by strain measurements which were made on all the upper-tier eye-bars

anchoring one of the cables during its erection. These measurements were planned at the outset to check the distribution of the cable pull between the strands. Specially designed, manually-operated, 20-in. strain-gauges with a lever ratio of 10 to 1 and 0.0001-in. Federal dials were used, enabling stresses as low as 150 lb. per sq. in. to be observed. Gauge holes were drilled in the eye-bars on the top and bottom faces, directly on the middle line of the bars, in order to eliminate the effects of bending. Strain measurements were taken after every set of four strands had been adjusted into place, all bars then in place being measured each time. Measurements were also taken after the floor steel was erected and again after the concrete roadway slabs were placed. In this way nearly 2 500 strain measurements were made. Table 3 gives the percentage by which the stress in each strand was greater or less than the average measured in the strands for each set of measurements. A study of these results shows that each new set of four strands, as it was adjusted into place, unloaded some of its stress into the strands previously placed, the four lowest strands being stressed to as much as 40% more than the average by the time the cable was completed. The addition of the floor steel and the concrete slabs, as was to be expected, reduced the percentage variation in strand stress. Except in a few cases, the average measured stress agreed well with the calculated average for the stages of construction at which the measurements were made, this agreement being particularly close in the case of the last two sets. In the last set of measurements, which represents practically the present condition of loading on the bridge, the lowest strands had 20% more stress than the average. This is equivalent to a unit stress of 3 000 lb. per sq. in. in the eye-bars, or 10% of the allowable unit stress.

In additon to the strain measurements on the eye-bars adjacent to the strand shoes, measurements were also made, during various erection stages of the bridge, of the strain in some of the eye-bars embedded in the concrete. In the design of the anchorage steel it was assumed that full stress remained in the bars down to the anchorage girders, and all pin details and girder details were based on this assumption. It was realized that undoubtedly much of the stress would be removed from the eye-bars before it reached the girders, but in the absence of some definite knowledge of how much and how fast this relief of stress took place, it was not considered wise to take advantage of it in the design.

It was felt well worth the effort, however, to attempt to determine something of the stress behavior of the embedded anchor steel, and, accordingly, a series of remote reading, electric telemeters were installed for observing the strains in the eye-bars after embedment in the concrete. The middle line of the middle chain of eye-bars in one of the cables anchored in the mass concrete of the New York anchorage was chosen for the test. Eight telemeters were installed in pairs, one on the top and one on the bottom, at points near the ends of corresponding eye-bars in the two lower tiers, the uppermost point being outside the concrete during most of the construction of the bridge. The location of the telemeters is shown in Fig. 5.

TABLE 3.—PERCENTAGE OF VARIATION FROM THE AVERAGE OF MEASURED STRESSES IN ANCHORAGE EYE-BARS FOR ADJUSTED STRANDS OF ONE CABLE AT THE NEW YORK ANCHORAGE.

Strand No.	Set No.																
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16*	17†
1	+8	+16	+19	+25	+22	+29	+31	+31	+36	+36	+34	+37	+39	+39	+40	+20	+19
2	−6	+12	+15	+18	+19	+22	+25	+23	+28	+28	+29	+32	+32	+32	+35	+19	+19
3	−1	+10	+18	+22	+20	+27	+29	+29	+34	+34	+34	+36	+36	+35	+36	+21	+18
4	−1	+19	+19	+25	+26	+25	+31	+30	+37	+35	+35	+36	+41	+39	+43	+23	+21
5	.....	− 4	− 4	+ 1	+ 6	+ 8	+12	+10	+12	+11	+12	+13	+12	+12	+12	+ 7	+ 6
6	.....	−19	−10	− 6	− 4	+ 1	+ 3	0	+ 3	+ 1	+ 4	+ 5	+ 7	+ 6	+ 9	+ 8	+ 9
7	.....	−16	−10	− 5	− 3	+ 1	− 1	+ 1	− 1	+ 4	0	+ 4	+ 2	+ 5	+ 6	+ 4	+ 4
8	.....	−21	−13	− 7	− 6	0	+ 1	− 2	+ 6	+ 2	+ 2	+ 4	+ 5	+ 6	+ 6	+ 6	+ 6
9	.....	.....	− 6	− 2	+ 1	+ 4	+ 5	+ 5	+ 5	+ 8	+ 7	+ 8	+ 9	+10	+ 8	+ 6	+ 5
10	.....	.....	− 5	0	+ 3	+ 8	+ 9	+ 8	+ 8	+ 8	+ 9	+11	+11	+12	+10	+ 6	+ 3
11	.....	.....	−14	−10	− 6	− 2	− 6	− 2	− 3	+ 1	− 1	+ 3	+ 2	+ 2	+ 4	+ 2	+ 3
12	.....	.....	− 5	+ 1	+ 1	+ 8	+ 8	+ 7	+11	+ 8	+ 9	+11	+10	+15	+10	+ 7	+ 6
13	.....	.....	.....	−11	− 7	− 5	− 5	− 3	− 1	0	− 2	0	+ 4	+ 3	+ 3	+ 3	+ 3
14	.....	.....	.....	−17	−10	−11	− 7	− 9	− 6	− 5	0	− 2	0	− 1	− 1	0	− 1
15	.....	.....	.....	−13	− 9	−10	− 7	− 8	− 5	− 2	− 5	− 2	− 3	− 1	0	− 3	− 2
16	.....	.....	.....	−21	−12	−12	− 7	−10	− 5	− 6	− 7	− 3	− 1	− 3	− 3	− 3	− 3
17	.....	.....	.....	.....	−11	− 7	− 9	− 6	− 7	− 3	− 6	− 3	− 3	− 1	− 2	+ 1	+ 3
18	.....	.....	.....	.....	−13	−14	− 9	− 9	− 5	− 6	− 6	− 5	− 5	− 3	− 2	− 1	0
19	.....	.....	.....	.....	− 7	− 7	− 8	− 4	− 5	− 2	− 5	− 2	− 1	+ 2	− 1	+ 1	+ 3
20	.....	.....	.....	.....	−12	−14	− 8	−10	− 6	− 6	− 6	− 5	− 3	− 4	− 4	+ 1	+ 1
21	.....	.....	.....	.....	.....	−13	−14	− 9	−10	− 5	− 6	− 6	− 8	− 7	− 5	− 5	− 5
22	.....	.....	.....	.....	.....	−10	− 8	− 5	− 3	− 4	− 3	− 5	− 4	− 2	− 2	− 5	− 5
23	.....	.....	.....	.....	.....	− 6	−12	− 3	− 4	+ 1	− 2	0	− 4	− 2	+ 2	− 4	− 3
24	.....	.....	.....	.....	.....	−12	− 5	− 9	− 2	0	+ 2	+ 3	− 4	+ 2	+ 3	− 5	− 2
25	.....	.....	.....	.....	.....	.....	−11	− 7	−14	− 7	− 8	− 7	− 8	− 2	− 6	− 2	− 1
26	.....	.....	.....	.....	.....	.....	−12	−13	−11	−11	−13	−11	−11	− 6	− 9	− 5	− 3
27	.....	.....	.....	.....	.....	.....	−13	− 7	− 5	− 4	− 3	− 2	− 6	− 1	− 2	− 6	− 2
28	.....	.....	.....	.....	.....	.....	− 2	− 5	− 2	− 4	− 1	− 2	− 2	0	+ 3	0	0
29	.....	.....	.....	.....	.....	.....	.....	0	− 5	0	0	+ 1	+ 2	+ 4	+ 3	+ 3	+ 5
30	.....	.....	.....	.....	.....	.....	.....	− 7	−14	−12	−11	− 9	− 7	−12	− 9	− 7	− 2
31	.....	.....	.....	.....	.....	.....	.....	0	− 7	− 3	− 1	0	− 1	+ 2	+ 1	− 4	0
32	.....	.....	.....	.....	.....	.....	.....	− 9	− 8	− 5	− 4	− 3	− 3	− 5	+ 3	− 2	+ 1
33	.....	.....	.....	.....	.....	.....	.....	.....	−20	−11	−13	−12	−12	−11	−15	− 7	− 2
34	.....	.....	.....	.....	.....	.....	.....	.....	−18	−12	−16	−14	−13	−16	− 9	−11	− 3
35	.....	.....	.....	.....	.....	.....	.....	.....	−18	−16	−18	−16	−16	−15	−12	− 9	− 6
36	.....	.....	.....	.....	.....	.....	.....	.....	−10	− 8	− 7	− 7	− 5	− 7	− 2	− 5	− 3
37	.....	.....	.....	.....	.....	.....	.....	.....	.....	− 9	− 7	− 7	− 4	− 2	− 6	− 1	− 6
38	.....	.....	.....	.....	.....	.....	.....	.....	.....	− 7	− 4	− 8	− 6	− 9	− 6	− 1	− 3
39	.....	.....	.....	.....	.....	.....	.....	.....	.....	−10	− 6	− 8	− 6	− 4	− 6	− 4	− 7
40	.....	.....	.....	.....	.....	.....	.....	.....	.....	−10	− 4	− 8	− 6	−10	− 7	− 3	− 5
41	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	− 3	− 9	− 3	− 3	− 3	− 2	− 5
42	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	− 7	−13	− 7	− 7	− 7	− 3	− 3
43	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	− 8	−15	−13	−14	−16	−12	−10
44	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	− 5	− 9	− 5	− 9	− 7	− 5	− 5
45	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	− 7	−10	− 7	− 6	− 4	− 4
46	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	− 7	− 6	− 7	− 7	− 1	− 2
47	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	− 9	−10	− 8	− 8	− 3	− 5
48	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	− 5	− 2	− 3	− 4	− 2	− 4
49	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	+ 3	+ 3	+ 2	+ 2	+ 1
50	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	− 1	− 2	− 3	+ 1	− 1
51	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	+ 1	− 1	− 4	− 2	− 2
52	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	− 1	− 2	− 5	0	− 3
53	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	− 6	− 5	0	− 4
54	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	− 6	− 2	+ 2	0
55	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	− 6	− 8	− 3	− 5
56	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	−11	− 8	− 5	− 6
57	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	−11	− 4	− 5
58	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	−18	− 8	− 8
59	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	− 8	− 5	− 6
60	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	− 8	− 2	− 4
61	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	− 4	− 1	− 3

* Set No. 16, floor steel erection completed.

† Set No. 17, concrete floor-slabs in place.

The double-cartridge, carbon-pile telemeter works on the principle that a change in the compression in a stack of carbon disks, which are part of an electric circuit, is accompanied by a change in the electrical resistance of the stack.[8] The telemeters themselves, although they are compensated and, therefore, independent of temperatures, are affected by humidity or moisture. Special precautions were taken therefcre to keep the instruments dry. Each was tap-screwed directly to the top or bottom of the eye-bar as the case might be, and then a cast bronze case or cover was placed over it and independently secured to the eye-bar using a gasket between the case and the steel. Where the wires emerged from the case elastic gum and water-proofing compound stuffing was used. Before concreting, the entire assembly was wrapped with water-proofed tape, and painted. Small resistance thermometers were also attached to the eye-bars in the same cases with the telemeters for observing temperatures.

The measurements indicate quite clearly that, except for the relatively small stresses from temperature or shrinkage of the setting and curing concrete, none of the telemeters embedded in the concrete has shown any stress in the eye-bars. In other words, if the reliability of the instruments is

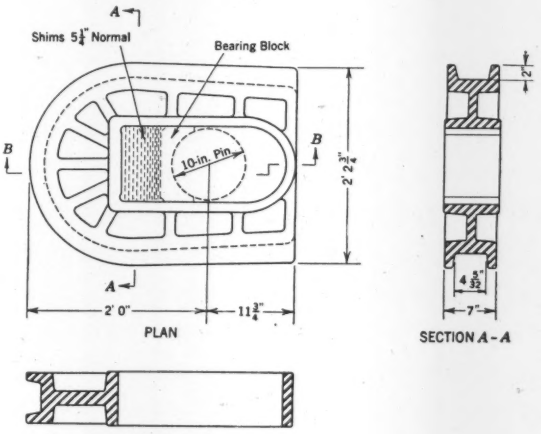


FIG. 10.—DETAILS OF STRAND SHOE.

accepted, the stress is dissipated in bond to the concrete in, at most, a length of 25 ft. in the line of eye-bars investigated. There is no reason to doubt that the same is true of the stress conditions in all other bars. If these bars were carrying their full load they would at present be under a stress of 16 000 lb. per sq. in. The telemeters are still (1932) being checked periodically in order to detect any sudden or gradual pick-up of stress, if it should occur, and can be read when any additional dead load is placed on the bridge.

[8] "Recent Developments and Applications of the Electric Telemeter," by O. S. Peters. *Proceedings*, Am. Soc. for Testing Materials, 1927, and *Technologic Paper No. 247*, National Bureau of Standards, U. S. Dept. of Commerce.

Strand Shoes.—The strand shoes used for terminating the cable strands and anchoring them to the anchorage eye-bars, are steel castings of the conventional design. (See Figs. 10 and 11.) Their design was based on functional demands rather than on strength requirements. Their major dimensions were determined by "scaling up" from the corresponding dimensions of the strand shoes used on the Delaware River Bridge[6], for which full-sized tests with strain measurements were made. Afterward, a somewhat extended elastic stress analysis was applied to the design, and this indicated stresses in the shoe well within allowable limits. The detail of the shoe differs in one respect from conventional design in that the side walls of the groove restraining the strand wires are beveled rather than vertical. Although all shoes were made alike in this regard this was done primarily for the shoes in the New Jersey anchorage. At that point the strands are forced by design to bend laterally through a small angle as they leave the shoes. As each half of the strand gradually departs from the curved bearing surface of the shoe, the bevel on the sides of the groove furnishes an easement curve for the lateral deflection of the wires.

Provision was made in the design for a longitudinal adjustment of the shoe of $10\frac{3}{4}$ in. to allow for any errors in the lengths of the guide wires, quite naturally to be expected, and for any errors in calculation of the lengths of the different strands, which, in this bridge, was an unusually difficult computation. Proof of the adequacy of this allowance for adjustment is the fact that the maximum average thickness of shims actually used at the two ends of a strand was 10 in., the minimum was 3 in., the grand average being 6 in., as compared with the $5\frac{1}{4}$ in. assumed in the design for the normal thickness.

Anchorage Saddles.—Reference has been made to several functions of the anchorage cable saddles to change the direction of the cables both horizontally and vertically and to act as splay castings which will restrain and guide the individual strands to their respective anchorage eye-bars. A more detailed idea of the design of these saddles may be had from a reference to Figs. 12 and 13. Each saddle is a single steel casting weighing about 21 tons. Bearing is on a nest of 12-in. carbon-steel segmental rollers on a 4-in. rolled-steel slab which rests directly on the concrete. This roller nest is keyed in the usual manner to the base of the saddle and to the slab, and is provided with side-bars and stop-plates to assist in erection. The rollers will operate dry and are protected by a dust-guard frame which is bolted to the saddle through slightly oversized holes in the frame to permit it to slide on the slab.

These rollers are required for taking up the changes in length of the back-stay cable and anchorage eye-bars due to changes in temperature and stress. The maximum calculated motion of the saddle riverward from normal, due to simultaneous extremes of temperature and live load, is 1.1 in., while the maximum shoreward motion from normal is 0.4 in. This latter dimen-

[6] Final Rept. of the Board of Engrs. to the Delaware River Bridge Joint Comm., June 1, 1927.

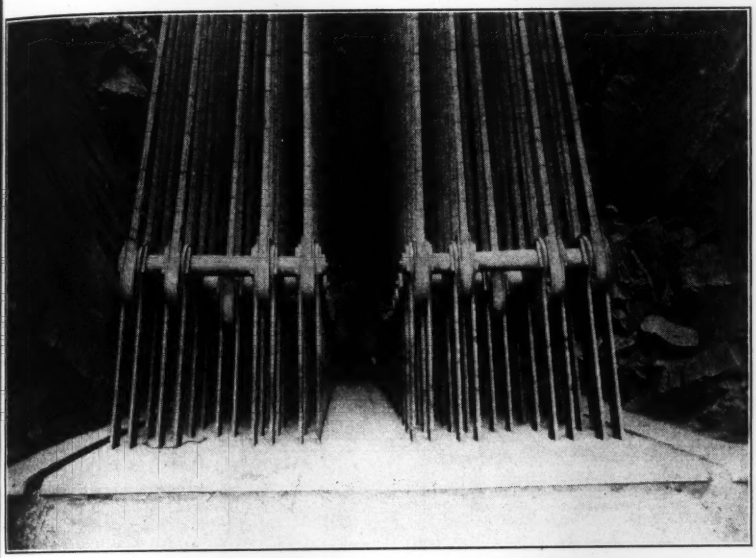


FIG. 11.—STRAND SHOES IN NEW JERSEY ANCHORAGE.

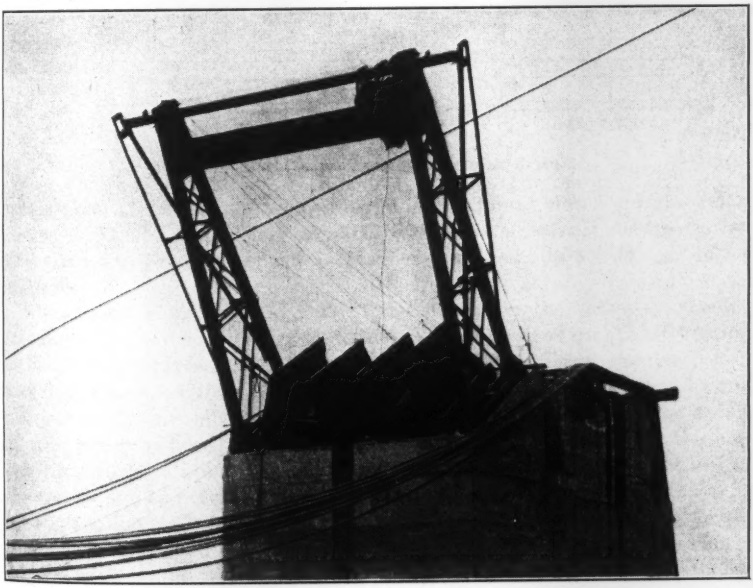


FIG. 12.—ANCHORAGE CABLE SADDLES, NEW YORK.

sion is exceeded temporarily during the construction of the cables when the saddle has to be set shoreward about 2.5 in. from normal position, which represents approximately the total back-stay stretch resulting from the addition of all the dead load of the bridge in its completed condition. At both the New York and New Jersey anchorages the plane of the rollers was established in such a position that the pulls in the side-span and back-stay cables would be equal. The small break in vertical angle of the cable as it passes

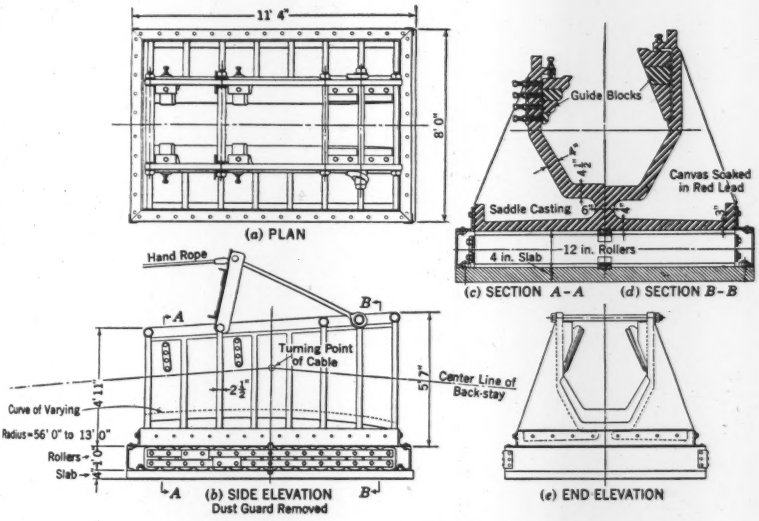


FIG. 13.—DETAILS OF ANCHORAGE CABLE SADDLE.

through the anchorage saddle results in a maximum thrust on the roller nest of 10 200 000 lb., or only one-fifth of the cable reaction on the tower. The maximum pressure per linear inch of the rollers is 680 lb. per in. of diameter.

On account of the vertical flare of the strands in the back-stay, the top strands, with their small angular deflection, have a short length of bearing on the saddle, whereas the bottom strands, with their large angular deflection, have a long length of bearing on the saddle, intermediate strands having intermediate lengths of bearing. At the upper or riverward end of the saddle all strands are in bearing while at its lower or shoreward end only the bottom strands are in bearing. Therefore, in order to equalize the pressure of the strands on the saddle along its length it was not constructed with a circular curve, but with a curve of varying radius, the greatest radius, 56 ft., being at its upper end and the smallest, 13 ft., at its lower end. To obtain uniform pressure the radius of curvature must be proportional to the number of strands in bearing; but the number of strands in bearing at various points along the saddle depends on the shape of the curve, being inversely propor-

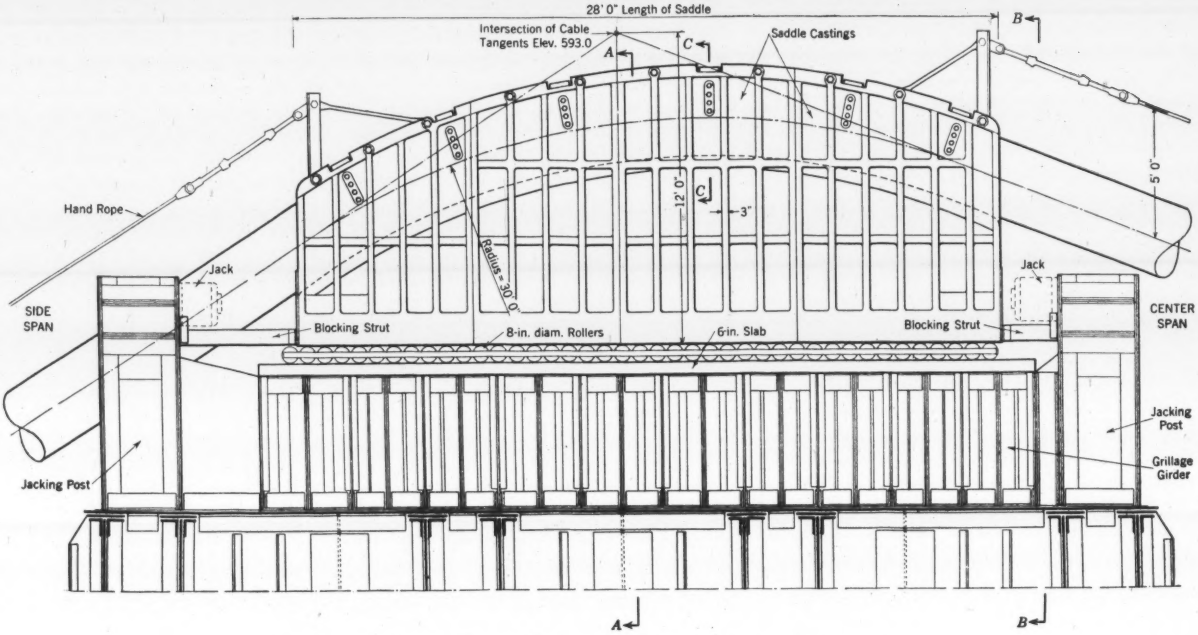


FIG. 14.—ELEVATION, TOP OF TOWER (SEE, ALSO, FIGS. 15 AND 16).

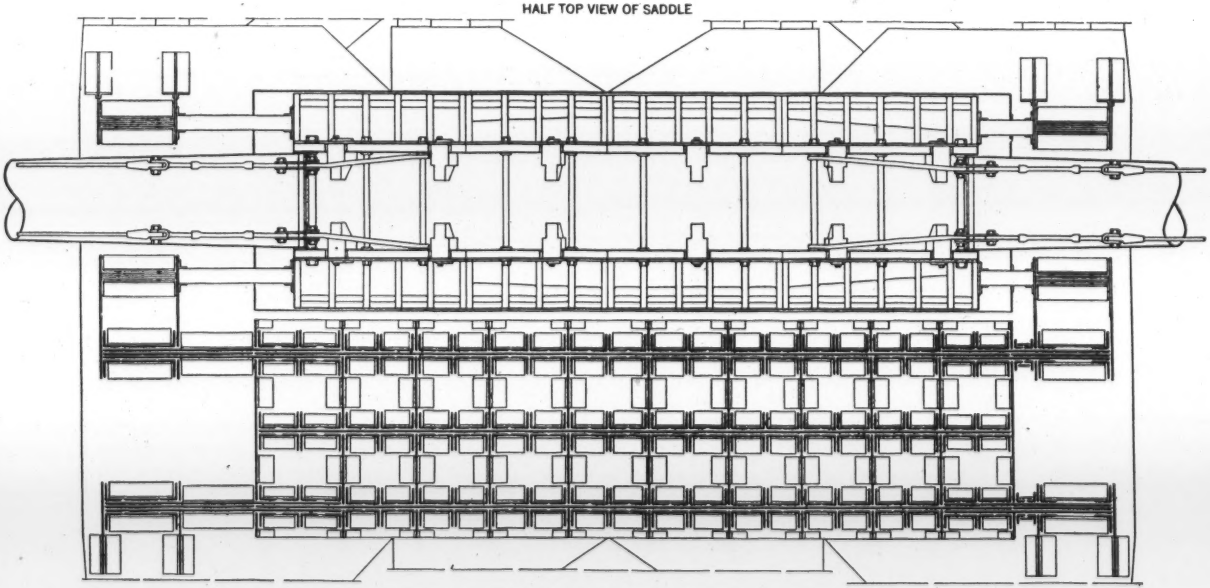
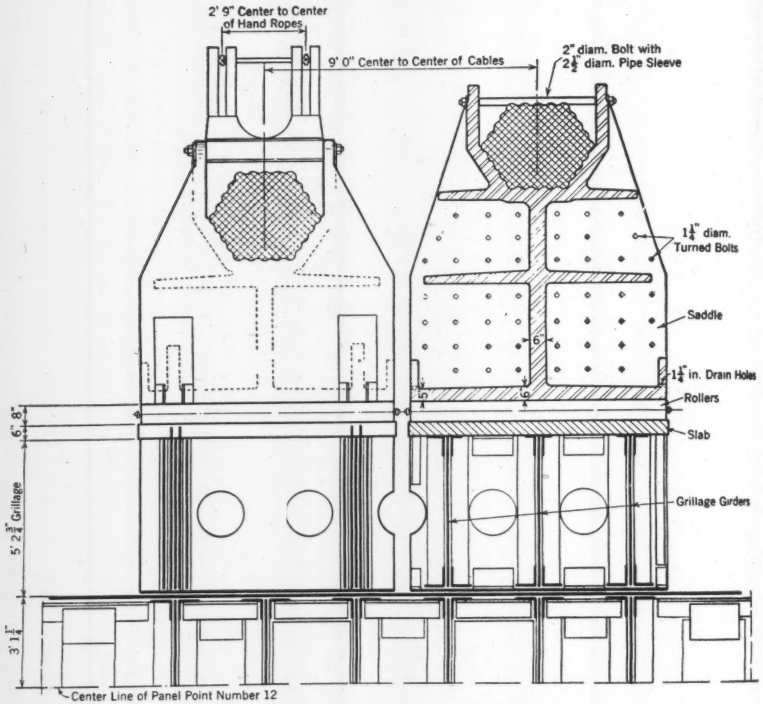


FIG. 15.—HORIZONTAL SECTION AND TOP VIEW, TOP OF TOWER (SEE, ALSO, FIGS. 14 AND 16).



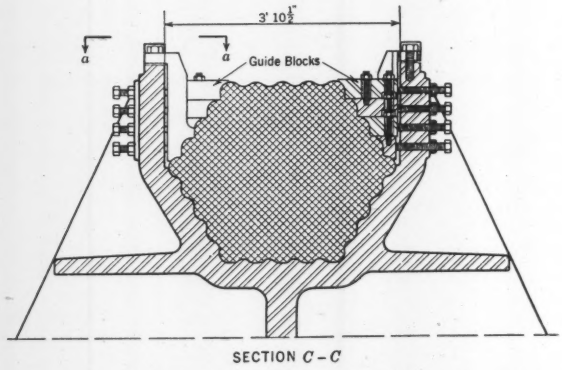


FIG. 16.—TYPICAL SECTIONS, TOP OF TOWER (SEE, ALSO, FIGS. 14 AND 15).

tional to the deflection angle of the curve. The curve, therefore, must be such that the radius varies inversely with its deflection angle. This curve is the tractix, or involute to the catenary, and by assigning a definite value to its length, and definite values to the terminal radii at each end, the parameter of the particular tractix required was determined. The bottom inside surface of the saddle was cast to this curve and, in this way, a uniform pressure on the rollers was obtained.

For about 3 ft. at the shoreward end of the saddle the sloping side surfaces flare outward in circular arcs, permitting the cable strands to splay laterally. At the top the strands are restrained and gradually led off laterally by curved wedged-shaped steel blocks. As the flaring of the strands would have made grooving of the saddle difficult, grooves were not used, and the surfaces were cast smooth and not machined.

Tower Saddles.—Where the cables pass over the tops of the towers they are supported in large saddles (Figs. 14, 15, 16, and 17). Each saddle, which is comprised of four steel castings bolted together, is 28 ft. long, 8.5 ft. wide, and 10.8 ft. high, and weighs 180 tons. The saddle rests on a bed of forty-one 8-in. steel rollers bearing on two 6-in. rolled steel slabs, butt ended together. These slabs are tap-bolted from beneath to the top flanges of steel grillage girders. There are three grillage girders, 5 ft. $1\frac{3}{4}$ in. deep, with closely spaced transverse diaphragms, under each saddle. The grillages serve to keep the cables clear of the top of the tower and assist in distributing the load to the tower columns. Jacking posts are connected to the ends of grillages.

As previously mentioned the rollers are provided not for the purpose of permitting the saddles to roll on the tops of the towers for live load and temperature saddle motions, but to facilitate erection and to permit future adjustments. Under ordinary conditions the saddles will be blocked against the jacking posts of the grillages in such a way as to prevent relative motion between saddle and tower, and the saddle motions will be participated in by the tower; that is, the tower will be forced to deflect longitudinally.

TABLE 4.—SADDLE MOTIONS AND CABLE DEFLECTIONS

Item of dead load	MOTION OF SADDLES, IN INCHES		INCREASE OF CENTER-SPAN SAG, IN FEET	
	Additional	Total	Additional	Total
Upper deck steelwork	....	8.00	...	7.0
Concrete slabs—side roadways	5.25	13.25	4.3	11.3
Concrete slab—center roadway	2.00	15.25	1.9	13.2
Lower deck complete	7.75	23.00	6.8	20.0

During the construction of the cables there is no appreciable movement of the saddles, except that due to temperature, because each strand, as it is added to the cable, has the same stress as in the completed cable. The addition of the dead load of the floor during its erection, however, increases the stress in the cable, and the consequent stretching of the side-span cables causes the saddles to move riverward. The magnitude of these saddle motions for the various increments of dead load are given in Table 4, which also

gives the increase of the sag of the cables in the center span caused by the combined effect of the stretch of the center-span cables and the movement of the saddles.

As it was considered desirable to have the saddles in their normal position over the center line of the tower, and thus avoid eccentricity of load, when the bridge is completed in its final stage with the lower deck, the saddles were set on the rollers 23 in. shoreward from this position during the construction of the cables. Rather than permit the saddles to roll riverward as the dead load was added (which would have been an uncertain action on account of the rolling friction and the flexibility of the towers), the saddles were kept blocked to the towers during the erection of the floor. Consequently, the tower tops were deflected riverward as the saddles moved, and at intervals during the floor erection the blocking was temporarily removed and jacks were operated between the saddles and the towers to roll the tower tops shoreward and thus avoid an undesirable amount of bending of the towers. This jacking was done twice during steel erection and again after the side roadway slabs were poured. In the last jacking operation the tower tops were rolled to a position, 2 in. shoreward from a vertical position, and the permanent steel blocking was placed. This position was selected so that the towers would be vertical when the upper deck is completed by the paving of the center roadway, and the saddles move 2 in. riverward.

At such future time as the lower deck is constructed the saddles will move riverward the remaining $7\frac{3}{4}$ in., to their final position, and the tower tops will be deflected that amount. The tower tops are then to be jacked back to a vertical position, and beyond to a position $5\frac{1}{2}$ in. shoreward, in order to minimize the stresses in the tower. The bending of the tower due to the lengthening and shortening of the side-span cables by live load and temperature is greater in the riverward, than in the shoreward, direction, and the riverward bending occurs with a greater vertical load on the tower. Consequently, to minimize the combined stress from vertical load and bending, the maximum stress on the riverward and shoreward sides of the tower must be equalized, which is accomplished by having the tower bent shoreward for the condition of dead load and normal temperature.

Thus, it is seen that the rollers have functioned only three times during the construction of the upper deck and will function again only when the position of the tower tops is adjusted after the lower deck is constructed.

Reference to Fig. 14 will show the very simple form of roller nest used. The usual side-bars are included, but there is no provision for keying the rollers to either the saddle or the slab, the pressure itself being considered ample insurance of maintaining the alignment.

The large jacking posts that are attached to, and form a part of, the grillages at either end, are designed to withstand the maximum horizontal tower reactions and the maximum jacking forces required during any stage of construction of the bridge. Finished bosses have been made on the ends of the saddles to receive the concentrations from the jacks.

The maximum reaction on each saddle is 55 600 000 lb. This force requires a long bearing surface between the cable and the saddle, and must

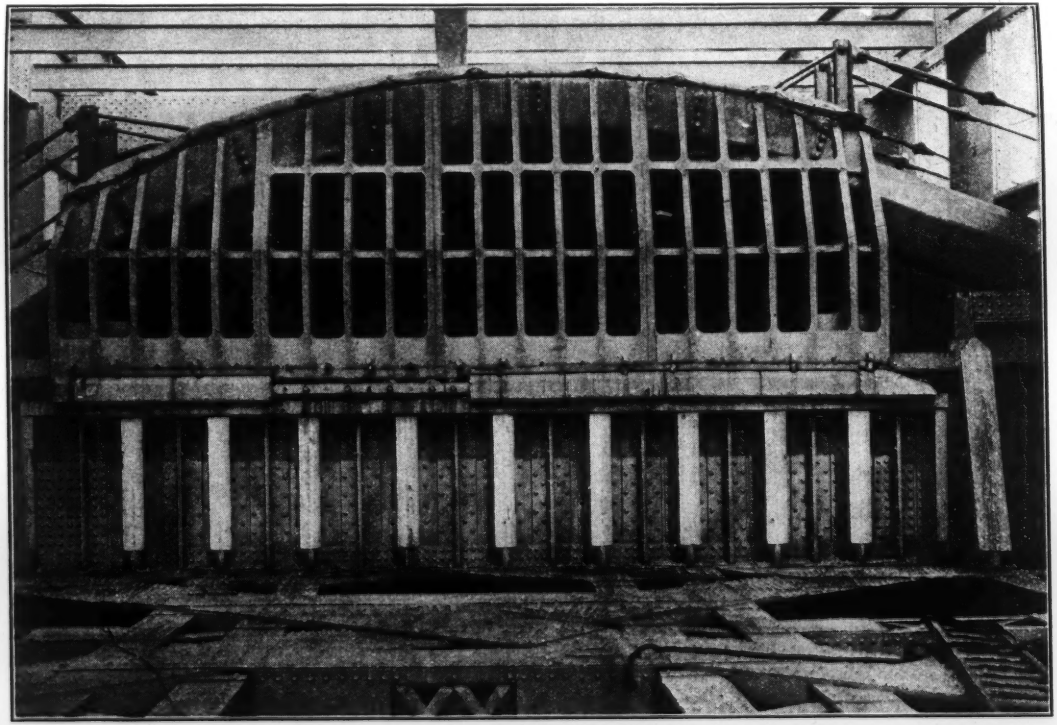


FIG. 17.—TOWER SADDLE, GEORGE WASHINGTON BRIDGE.

be distributed more or less uniformly to the tops of the four inside tower columns by means of the saddles and grillage girders. The girders, including the jacking posts, were made to extend the full width of the tower, and the saddles were made long enough to permit a 30-ft. radius for the cables, and still allow room between the ends of the saddles and the jacking posts for the jacks and the necessary saddle motions. The cables enter the saddles at angles with the horizontal of about 33° on the side-span side and 20° on the center-span side, resulting in unsymmetrical saddles.

This inequality of angle, results in a corresponding inequality in tension in the cable at the two ends of the saddle, as previously mentioned, the tension being 12% greater on the side-span end. The difference in tension is absorbed by friction between the cable and the saddle, the amount of friction required being 13% of of the vertical load. This is increased to 15% under the most adverse conditions of live load and temperature.

For the cable to be safe against slipping in the saddles, the actual amount of friction available must exceed the amount required. Friction tests of wires, made in connection with the Delaware River Bridge,[6] indicate that a friction between wires and saddle of 20% can be counted on and this figure was used in the design, especially as it related to conditions during erection. In order to obtain the maximum friction, the inside bearing surfaces of the saddles were roughened after they were machined, and instead of a weather protection of grease they were given a coat of paint. It was expected that, if anything, the paint would increase the friction, but experiments made in the field after the first strand had been set in the saddle, showed that this was not the case and, consequently, the paint was removed.

While the first strands were being adjusted under low temperature, some of them slipped in the saddles, and, by making the necessary field observations and calculations for the conditions at the time, this slipping was used to ascertain the actual coefficient of friction. It was found that slip occurred when the unbalanced pull was 23% of the vertical load for a painted surface, and 30% for a surface from which the paint had been removed, which showed that the assumed value of 20% for the friction of a strand in the saddle was conservative.

The completed cable has a greater resistance to slip than that due merely to friction on the saddle. Before the cable can slip, it must bend and unbend as a whole at the two ends of the saddle, and for this to occur all the wires in the cable must slip on each other throughout the length of the saddle, overcoming the friction between them. The internal friction of the wires thus tends to lock the cable into its curved shape in the saddle and adds greatly to the factor of safety against slip.

Regarding the more detailed design of the tower saddles involving the determination of thicknesses of metal used, little can be said other than that it was based upon a conservative application of general and simple rules of design. The actual distribution of stress throughout such a casting is too indeterminate to permit of an application of the law of elasticity. The details of the saddles, such as tie-rods and wedge-blocks, were more or less conventional.

However, in dimensioning the cross-section of the inside surfaces of the saddle and the shape of the strand grooves, an allowance was made for the tendency of the strands to flatten under their own weight, when placed in the saddle, in spite of their seizing. Consequently, instead of assuming truly circular strands assembled to form a regular hexagon, it was considered that each strand would flatten to the shape of an ellipse, 4 by 5 in., and that the entire cable would take the form of a flattened and elongated hexagon.

Cable Bands.—The cable bands, on which the wire-rope suspenders bear, are in the form of two semi-circular steel castings bolted together in such a manner that they clamp the cable tightly between them. Each band carries in grooves cast in the outside periphery the bights of two suspender ropes. Fig. 18 shows the largest band, which is used at all panel points in the side

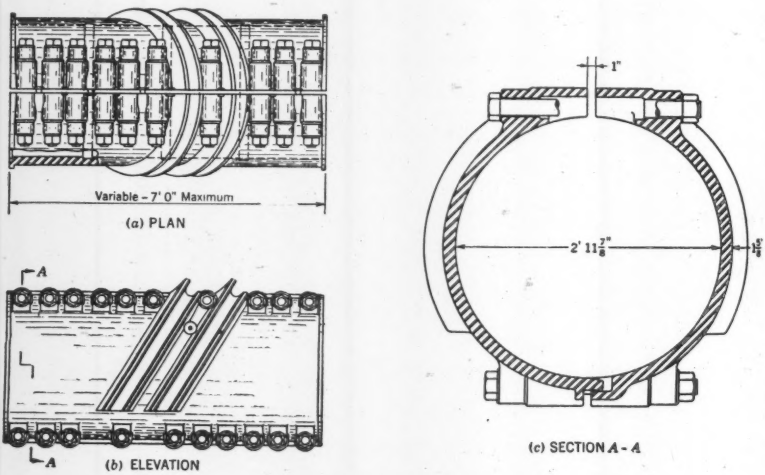


FIG. 18.—DETAILS OF CABLE BAND.

spans. Smaller bands of varying lengths are used in the center span, and are of similar design, except for the number of bolts and the inclination of the suspender grooves.

Since the bands at all panel points except the one at the middle of the center span are on sloping parts of the cables, they must grip the cables tightly enough to prevent them from slipping downward under the pull of the suspenders. The total clamping force provided is calculated for any particular band by dividing the component of the maximum suspender pull that is parallel to the cable by the allowable coefficient of friction. The component of suspender pull acting normal to the cable is not considered in the design as effective in creating friction. The required clamping force divided by the allowable stress in the bolts gives the required number of bolts. Heat-treated, turned bolts, $2\frac{3}{8}$ in. in diameter, were used at an allow-

able stress of 100 000 lb. per bolt, equivalent to a unit stress of 29 000 lb. per sq. in. of net section. A coefficient of friction of 15% was assumed in determining the necessary clamping forces.

The determination of what constitutes a safe allowable coefficient of friction between the bands and the cables is a mooted point. Laboratory tests of the ultimate resistance to slipping were made by the Delaware River Bridge Joint Commission,[8] on a band clamped to a sample section of the 30-in. cables. These tests showed an average coefficient of friction of about 60 per cent. It is believed that the local contraction in diameter of the cables due to the relatively greater compaction under the band than elsewhere is accountable for this high value. Tape measurements of circumferences of the cables of the George Washington Bridge for short distances adjacent to the bands, verified this relative swelling of the cables at both ends of the bands. These measurements showed a cable diameter appreciably less at the immediate ends of the band than at points 6 to 18 in. away. With such conditions existent, a 60% ultimate value of friction seems reasonable, fully justifying an allowable value of 15 per cent.

To insure, further, the development of friction between the bands and the cables the inside machined surfaces of the bands were specified to be finished to a definite degree of roughness (16 transverse tool cuts per inch). The use of grease on these machined surfaces for protection was discarded in favor of the application of one shop coat of paint.

The thickness of the main shell of the bands was made 1⅝ in. The shell is subjected not only to the ring tension developed in tightening the bolts, but also to considerable bending stress because the bolts are eccentric to the shell, and because the band must deform slightly from its truly circular shape in the process of squeezing the cable to fit it. It was necessary for the shell to have a margin of strength to take bending stresses and, at the same time, to be thin enough to stand some deformation.

During construction, while floor steel was being erected, a longitudinal crack developed in one of the cable bands in the center span. This crack, at the mid-depth of the band, extended about 14 in. in from one end and was considered serious enough to warrant replacement, which was made before the addition of the concrete slabs. A series of strain measurements were made on the new band to show the actual stresses as the bolts were tightened and during the addition of subsequent floor loads. The circumferential strains on the outside periphery of the main shell indicated stresses well within the allowable, but the strains in the extreme outside fibers of the end ribs of the band showed stresses above the yield point of the material. Longitudinal strains due to beam action from the suspender concentrations indicated that these stresses were very small. Fig. 19 shows the results of these measurements. It was evident that the additional stiffness provided by the end ribs resulted in an overstress in the latter as the band was deformed in squeezing the cable into a circular shape. Inasmuch as this overstress was local, and was immediately relieved by a redistribution of stress on adjoining sections, it was not considered harmful; a close inspection of all the remaining bands failed to disclose any further cracks either at this time or later.

It is quite possible that a local defect may have caused the crack, although none was detected in a superficial examination or chemical analysis of borings.

In accordance with the usual practice the bands were designed with the joints between halves placed vertically; that is, with the bolts in a horizontal position. A nominal gap of 1 in. was allowed between flanges for clearance

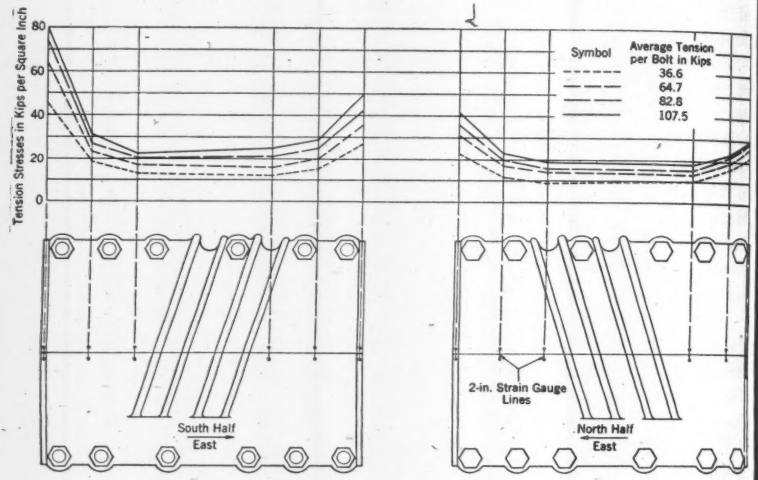


FIG. 19.—MEASURED STRESSES IN CABLE BAND.

and caulking. Circumferential grooves were provided on the inside surfaces of the longer bands to give clearance for the permanent seizing of the cables at these points. The ends of the bands were counter-bored in the usual manner for terminating the cable wrapping, and for caulking. The bolts and bolt housings were made quite long, so that the bolts could be close to the cable and their eccentricity minimized. The use of long bolts also lessened the effect on bolt tension of differences in temperature of the cable and the band. All bolts were provided in the shop with holes for strain-gauge measurements which were relied upon for the final check of the required bolt tensions. Calibrated dynamometers, checked frequently with the strain-gauges, were used in tightening the bolts.

To determine the exact diameter of bore for the cable bands the contractor was obligated to construct a full-sized sample section of cable 10 ft. long, and to compact it in the same manner as he expected to compact the actual cables. This gave a diameter of $35\frac{7}{8}$ in., which value was used in the final details.

Special bands are used adjacent to all saddles for terminating the wrapping and for providing means of attaching housings for the unwrapped parts of the cables at these points.

Suspenders.—The suspenders were designed to carry to the cables the entire dead weight of the completed upper and lower decks and a local live

load distributed to them by the stiffening trusses. Each suspender consists of a single wire rope, 2⅞ in. in diameter, looped over the cable in grooves in the cable band, socketed at both ends, and attached directly to the floor-beam. Two of the suspenders are supported by each cable at a panel point, which makes a total of eight suspenders, or sixteen lines of rope per panel. The maximum length of suspender is 685 ft. and the minimum, 48 ft., and the total length of suspender rope in the entire bridge is 170 000 lin. ft.

At a point 5 ft. below the center line of the cable the two halves of each suspender are pulled in from the 42-in. diameter, in which they pass around the cable band, to a spacing of 14½ in. and are held together by an articulated tension tie, which is prevented from slipping down by zinc collars cast on the ropes. From this point to their connections on the floor-beams the ropes hang vertically. In this way, the amount of space occupied by the suspenders at the floor level is kept to a minimum. Fig. 20 shows the design of the articulated tension tie and its assembly on the ropes.

The calculated maximum stress in the two parts of a suspender is 280 000 lb., or 35 000 lb. per sq. in. Of this, 200 000 lb., or 72%, is due to dead load and the remainder to live load and temperature. The suspender ropes were specified to have a minimum ultimate strength of 1 200 000 lb. in two parts, over a sheave equal in diameter to the band. Therefore, a safety factor of more than four is provided in the suspenders.

Using live loads of varying length and intensity according to the design specifications it was found that the maximum stress in a suspender occurs with the extended live load of 4 000 lb. per ft. over the entire bridge combined with a partial load of 7 700 lb. per ft. over a length of 360 ft., and with the lowest temperature. For this condition the suspenders carry 81% of the partial live panel load, and all the dead and extended live panel load.

The specifications also required the suspender ropes to have a maximum stretch of 0.3 in. in a length of 100 in. under a pull of 200 000 lb. after pre-stressing, which was equivalent to a modulus of elasticity of about 17 000 000 lb. per sq. in. for the rope as actually made. A nominal diameter of 2¾ in. was specified, but not the total wire area. Nor was the minimum ultimate strength of the rope in a single part specified, except that it was to have the same ratio to the required strength, in two parts, over a sheave as that established by the earlier acceptance tests, if the contractor chose to make acceptance tests on straight ropes thereafter.

The contractor proposed and was permitted to use ropes 2⅞ in. in diameter, made up of 6 strands of 37 wires each and an independent wire rope center with 7 wires in each of its 6 outside strands and a 19-wire center strand. The individual wires in the rope are of varying size and strength in order to give a rope of high efficiency. Each rope has a cross-sectional metallic area equal to 4.05 sq. in. and a modulus of elasticity after pre-stressing of more than 18 000 000 lb. per sq. in. The acceptance tests gave an average breaking strength of 1 336 000 lb. for two parts over a sheave, and 778 000 lb. for a straight rope.

All the rope used for suspenders was first utilized by the contractor as footbridge cables and was pre-stressed before being measured to the long

lengths required for that purpose. In service in the footbridges the rope was permitted to be stressed as high as 230 000 lb., or 65% more than the maximum design stress of the suspenders. It was felt that this higher stress rather than harming the rope, would be advantageous as additional pre-stressing.

When the cable spinning was completed, the ropes were released from carrying the footbridges; then, successively, they were hung free under a condition of balanced horizontal tension at the towers, with a predetermined sag in the center span, and were measured off into the proper lengths for suspenders. A detailed program had been prepared giving the location along the ropes of every one of the suspenders to be cut from them, and the lengths to be measured. To calculate these lengths, the final dead load lengths, as determined from the final elevations of the cables and floor at each panel point, had to be corrected to take care of the difference between the final dead load stress in the suspenders and the stress in that part of the ropes from which each suspender was to be cut. A further correction of length was made for any differences that had been observed in the relative elevations of the four cables as erected. This schedule was revised from time to time, as the work progressed, to meet the unavoidable re-arrangements necessary in actual field operation.

The design did not contemplate the use of shims for adjusting the lengths of the suspenders. Instead it was believed that, with special care in the manufacture, measurements, and erection of the ropes, adjustments would become an unnecessary refinement. With this in mind the specifications required a uniform modulus of elasticity for the rope with a maximum variation of 10 per cent. To eliminate changes in length due to twists subsequent to measuring, paint marks were required to be placed along the ropes when they were being measured, and the contractor was also obligated to have these paint marks in line when the floor-beams were hung upon the suspenders.

As stated previously, the suspenders are socketed for attachment to the floor-beams. The sockets, shown in Figs. 21 and 22 are very simple in design. They are made of cast steel in the form of truncated cones, $7\frac{1}{2}$ in. to 9 in. in diameter, and $12\frac{1}{2}$ in. long, cored out in the usual way for the "broomed" wires and the zinc spelter. They are drilled near the bottom for two $\frac{1}{2}$-in. steel pins which project on the inside into the broomed wire ends and thus prevent the socket from turning on the rope or slipping down the rope during handling and before the erection of the suspenders. In as many cases as possible the same sockets that were on the ends of the ropes when they were in the footbridge cables were used without removal from the rope.

Cable Wrapping.—The cables are protected by a wrapping of soft, annealed, and double-galvanized steel wire, continuous between cable bands, the cables being painted before and after the wrapping was applied. No. 9 (B. W. G.) wire, with a diameter of 0.151 in. over galvanizing, was used. It has an ultimate strength of 1 100 lb. and a yield point of 600 lb. per wire. The contractor was not permitted to wrap the cables until after the dead load of the entire steel floor deck and concrete roadway and sidewalk slabs were

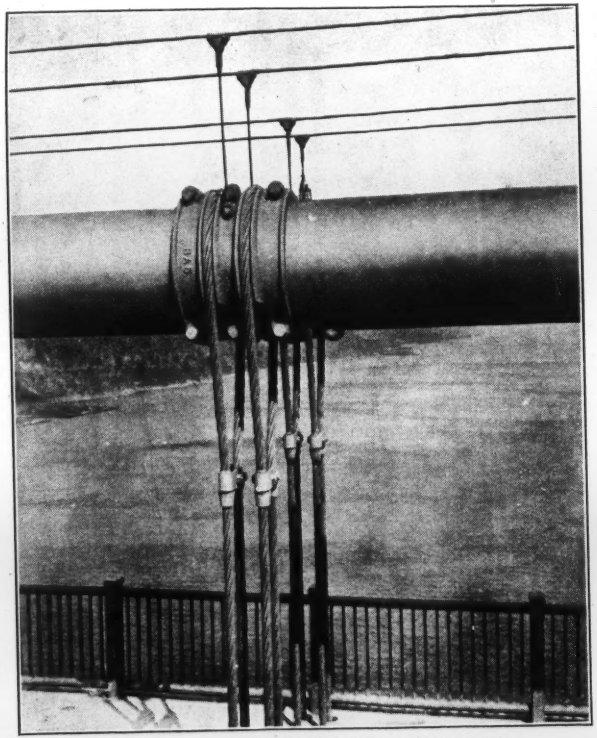


FIG. 20.—VIEW OF UPPER SUSPENDER CONNECTION.

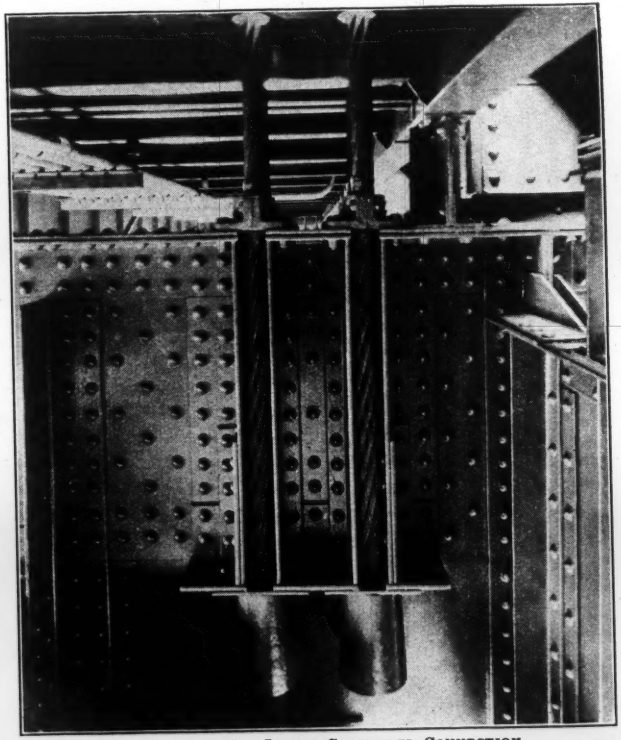


FIG. 21.—VIEW OF LOWER SUSPENDER CONNECTION.

in place, or, in other words, until the bridge was practically completed in its initial traffic condition with two-thirds of the final dead load in place.

Because of the unprecedented size of the cable, and because the addition of dead load in the future will increase the cable stress, tending to decrease

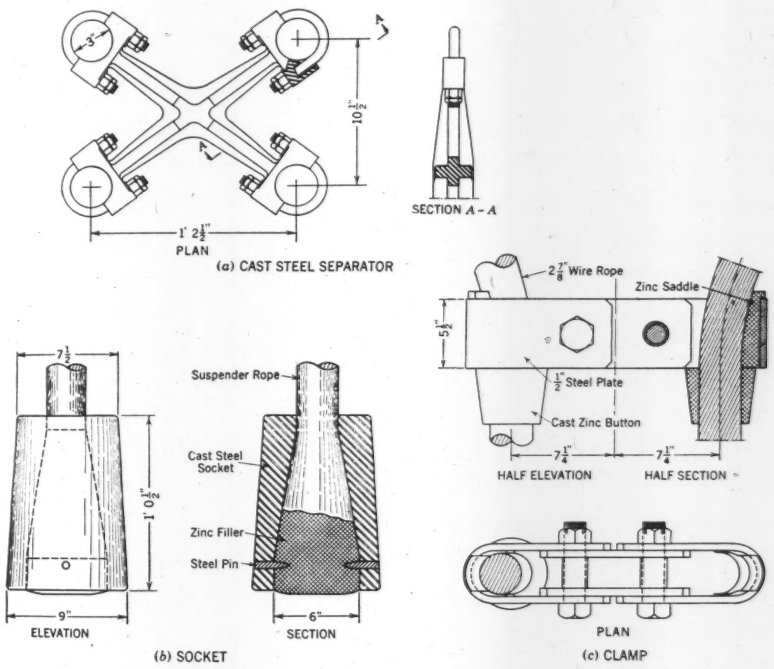
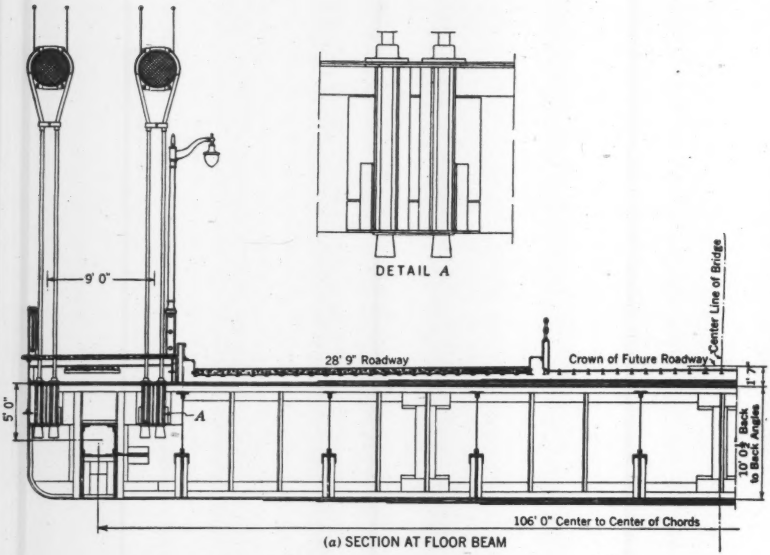


FIG. 22.—DETAILS OF SUSPENDER ROPE FITTINGS.

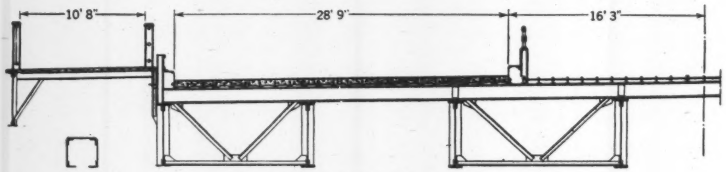
the diameter of the cable and loosen the wrapping, a tension of 400 to 500 lb., considerably greater than that used for previous large bridge cables, was specified.

DESIGN OF FLOOR SYSTEM

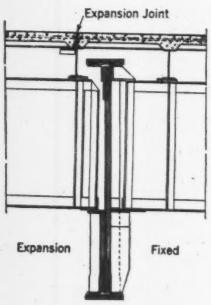
General Description.—The floor system of the upper deck, as constructed initially, is composed as follows (see Figs. 23 and 24): The main floor-beams, hung from the cables by the suspenders, are spaced 60 ft. apart. They support eight lines of roadway stringers and two fascia girders at their ends. On top of the stringers are placed transverse or secondary floor-beams, spaced about 5 ft. apart, which carry the concrete roadway slabs and four lines of steel curbs. The sidewalk slabs are supported by transverse beams, spaced 3 ft. 9 in. apart, which frame between the fascia girders and longitudinal beams fastened at 20-ft. intervals to the outside roadway stringers. Two



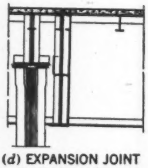
(a) SECTION AT FLOOR BEAM



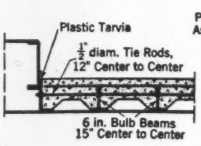
(b) SECTION NEAR CENTER OF PANEL



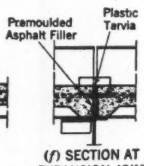
(c) STRINGER SUPPORTS



(d) EXPANSION JOINT IN FASCIA GIRDER



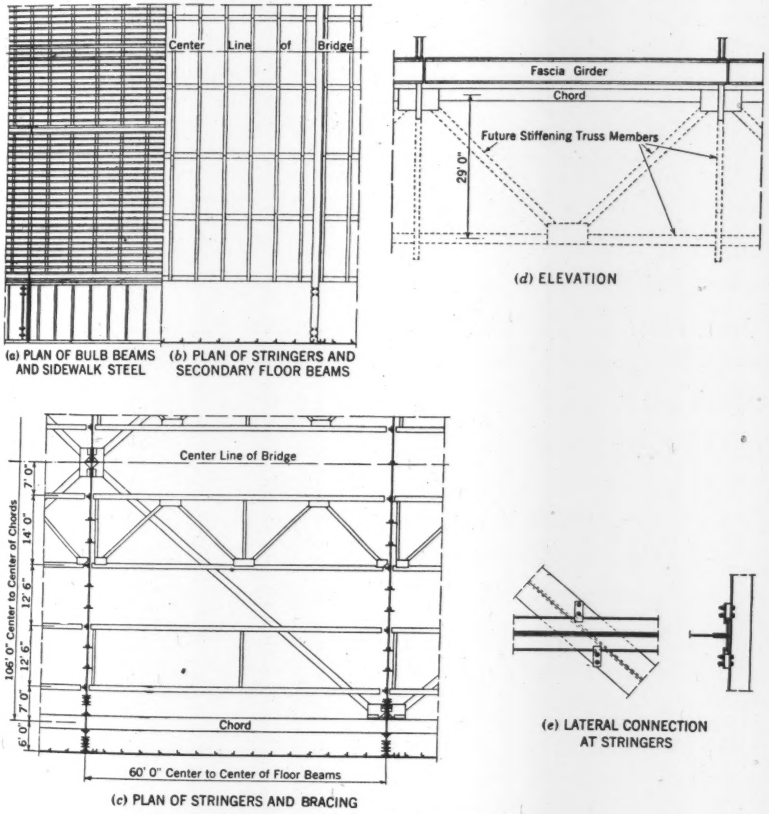
(e) ROADWAY SLAB



(f) SECTION AT EXPANSION JOINT

FIG. 23.—TYPICAL FLOOR DETAILS.

continuous wind chords, spaced 106 ft. apart, pass through holes in the floor-beams and, together with the main laterals placed just under the roadway stringers, form the wind trusses. These chords also form the top chords of the future stiffening trusses. The stringers and fascia girders are provided with expansion connections, and the roadway and sidewalk slabs with expansion joints on one side of each main floor-beam to allow for the angular



(a) PLAN OF BULB BEAMS AND SIDEWALK STEEL (b) PLAN OF STRINGERS AND SECONDARY FLOOR BEAMS

(d) ELEVATION

(c) PLAN OF STRINGERS AND BRACING

(e) LATERAL CONNECTION AT STRINGERS

FIG. 24.—TYPICAL FLOOR FRAMING.

distortions of the cables and floor from live load, only the wind chords and laterals being continuous past the floor-beams. Under the initial condition, without stiffening trusses, the angular motion centers at the wind chords. When the trusses are added the motion will center at their neutral axis. The articulation given by these joints will serve to eliminate participation by the floor structure in the stiffening truss action and will be an advantage in the erection of the lower deck.

The design for the lower deck, as shown in Mr. Ammann's paper[7] previously mentioned, is tentative and was developed only far enough to determine the design dead load, provision for future connections, and depth of stiffening trusses. The lower deck floor-beams will be suspended from those of the upper deck by means of structural hangers which will also form verticals in the future stiffening trusses. Rivet holes, temporarily filled with bolts, are provided for connection to the present floor-beams. In order to minimize the bending stresses in the hangers they are designed as slender members with pin connections to the floor-beams of the lower deck.

Main Floor-Beams.—The main floor-beams are plate girders, 10 ft. deep and 118 ft. long, each weighing 62.5 tons. They are supported at points directly below each cable by groups of four suspender ropes. The suspender sockets bear against shelf angles on the floor-beams, the reactions being taken by pairs of stiffeners fitted between the shelf angles and the top flange (see Fig. 21). The end floor-beam reaction is assumed to be distributed equally between the two groups of suspenders. This assumption is nearly correct because of the relatively great flexibility of the cables and suspenders compared to the rigidity of the floor-beam. The main material, splices, and suspender connection details are made of silicon steel, whereas carbon steel is used for stiffeners, fillers, stringer seats, and other details.

The maximum shear and the maximum bending moment for which the floor-beams are designed, are as follows:

	Maximum shear, in pounds	Maximum moment, in inch-pounds
Dead load	505 000	166 000 000
Live load	328 000	118 000 000
Impact	59 000	21 000 000
Total	892 000	305 000 000

The web-plate, which is spliced at the center and at the quarter-points, is 120 in. by $\frac{1}{2}$ in. Each flange consists of two angles, 8 by 8 by $1\frac{1}{8}$ in., two side plates, 18 in. by $\frac{1}{2}$ in., and four cover-plates, 20 in. by $\frac{3}{4}$ in. The floor-beams are given a camber of $1\frac{1}{4}$ in. at the center and 1 in. at the quarter-points.

Roadway Stringers and Secondary Floor-Beams.—The roadway stringers are plate girders varying in depth from 5 ft. 4 in. to 5 ft. 8 in. This variation is due to having the top flanges follow the crown of the roadway and keeping the bottom flanges, which support the wind diagonals, in the same plane. The flanges are made of silicon steel, and the web-plates (which are $\frac{3}{8}$ in. thick), and all details are made of carbon steel. Each flange consists of two angles, 6 by 6 in. by $\frac{1}{2}$ in., with a cover-plate, 14 in. by $\frac{1}{2}$ in., for the four center stringers, a cover-plate, 14 in. by $\frac{3}{4}$ in., for the next two outside stringers, and no cover-plate for the outside stringers.

Both the fixed and expansion ends of the stringers have seated connections to the floor-beam, the expansion end being provided with bolts in

[7] See p. 40, Fig. 20.

slotted holes. At the fixed ends, light connections are also made at the top to hold the floor-beam and the stringers from tipping. At the expansion ends and at the center of the panel, the stringers are held together in pairs with cross-frames, and a light horizontal bracing is used in the plane of the top flanges between the two central pairs of stringers.

The secondary floor-beams are 16-in., 43-lb. I-beams, spaced 5 ft. 2 in. apart, except at the expansion joint in the roadway slab where a 16-in., 50-lb. I-beam is used, with a spacing of 4 ft. 2 in. on either side. The beams follow the crown of the roadway, except that those in the middle bay under the curved part of the roadway crown are straight. They are riveted to the tops of the stringers and have beveled fillers under their ends to take care of the slope. The beams in the three middle bays, which are 14 ft. 0 in. wide, are made of silicon steel, and those in the four outer bays, which are 12 ft. 6 in. wide, are continuous over two spans and are made of carbon steel.

In designing the secondary floor-beams only 75% of the wheel loads was assumed to be carried by any one beam, the other 25% being distributed to adjacent beams by the floor-slab. A careful analysis of the action of the slab on its yielding supports showed that this assumption was conservative.

Roadway Curbs and Paving.—Four lines of steel curbs, of the double-step type (see Fig. 25), are supported by the secondary floor-beams. The two outside curbs are 90 ft. apart in the clear, but this space is broken up into three roadway spaces by the two intermediate curbs which are located so as to give a clear width of 28 ft. 9 in. for the two side roadways. At present (1932), only these two side roadways are paved.

The lower step of the curbs is 10 in. high and 12 in. wide. The upper step is 18 in. high and 7 in. wide. The outside curbs are close to the inner railings of the sidewalks which serve as additional barriers, while the intermediate curbs are provided with two-line pipe railings for the same purpose. The intermediate curbs are constructed so that when the center roadway is paved, the top step and railing can be removed and a plate added to the back of the lower step, in this way giving a 30 ft. 6-in. center roadway, separated from the side roadways by 10-in. curbs, 12 in. wide. If it is decided to have two roadways instead of three, the intermediate curbs can be moved from their present position and the two lower steps placed together at the center, for which arrangement connections to the secondary floor-beams are now provided, thus giving two 44-ft. roadways separated by a 10-in. curb, 24 in. wide. Another alternative is to remove the intermediate curbs entirely and have a single roadway 90 ft. wide.

The roadway pavement consist of monolithic reinforced concrete slabs, 28 ft. 9 in. wide between curbs, and 60 ft. long between expansion joints. The main reinforcing consists of 6-in., 14-lb. bulb beams running longitudinally, spaced 15 in. on centers, and riveted to the secondary floor-beams. The transverse reinforcing consists of $\frac{1}{2}$-in. tie-rods, 12 in. apart, located 4 in. above the bottom of the bulb beams, and $\frac{1}{2}$-in. reinforcing rods, 6 in. apart, placed on top of the beams and tied together with $\frac{1}{2}$-in. longitudinal rods spaced midway between the beams. The top of the slab is $2\frac{1}{2}$ in. above the top of

the bulb beams and the bottom is haunched 3 in. between the beams. The thickness of the slab is, therefore, $8\frac{1}{2}$ in. at the beams and $5\frac{1}{2}$ in. between the beams, the average thickness being 7 in.

The slabs have expansion joints, $\frac{3}{4}$ in. wide, over the secondary floor-beam on the expansion side of each main floor-beam. On the expansion side of this joint **T**-clips, fastened to the under side of every other bulb beam, project under the top flange of the secondary floor-beam to hold the slab down. A pre-moulded asphalt filler, $6\frac{1}{2}$ in. deep, is placed in the joint, and the upper 2 in. of the joint are filled with plastic tarvia. The details are shown in Fig. 23(*e*) and Fig. 23(*f*).

Bulb beams were selected for the main reinforcement in preference to the 6-in. **I**-beams originally called for, because they had a wider bottom flange for riveting to the beams below and because the small, rounded top flange, permissible in the design on account of the relatively small negative moments, made it possible to work the concrete around them in a satisfactory manner and avoided a flat steel surface near the top of the slab. The rolling of these bulb-beam sections had been discontinued and the rolls destroyed, but arrangements were made to have new rolls made, and it was decided to have the bulb beams for the unpaved center roadway also furnished and erected with their tie-rods. They serve as a protective covering to the otherwise open framing of the central roadway space prior to its being paved.

By arranging the main reinforcement beams longitudinally instead of transversely it was unnecessary to bend them to fit the roadway crown, and the heavy wheel loads were distributed to them with less shear in the concrete.

The lower part of the curbs consists of sections made up of 6-in. **Z**-bars and angles. If the intermediate curbs are removed from their present location these sections will remain, taking the place of bulb-beam reinforcing. This type of bar is also used instead of bulb beams at the middle of the center roadway space for supporting the curbs in case they are moved to that location.

The completed roadway will have a crown of $5\frac{1}{8}$ in., with straight slopes of $\frac{1}{8}$ in. to the foot, connected by a curve at the center, 16 ft. long. Cast-iron drainage scuppers with cast-steel gratings and pipes extending down to the bottom of the stringers are placed at 30-ft. intervals along the outside curbs. If the completed deck is divided into three roadways additional scuppers will be provided at the sides of the central roadway.

The roadway slab is designed for 18 000-lb. wheel loads plus 75% impact, and the unit stresses are comparatively low. The concrete was specified to have a strength of 4 000 lb. per sq. in. after 28 days. Actually, it had an average strength of 4 600 lb. per sq. in. The average weight of the slab is 97 lb. per sq. ft.

Sidewalks.—The sidewalk slabs are 3 in. thick and are reinforced with single mats of $\frac{3}{8}$-in. rods spaced 6 in. transversely and 12 in. longitudinally. They are supported by 8-in., 21-lb. **I**-beams running transversely and spaced 3 ft. 9 in. apart. As previously mentioned, these beams are connected to the fascia girders at their outer ends and the longitudinal beams at their inner ends. The fascia girders are light plate girders, 4 ft. $9\frac{1}{2}$ in. deep, made up

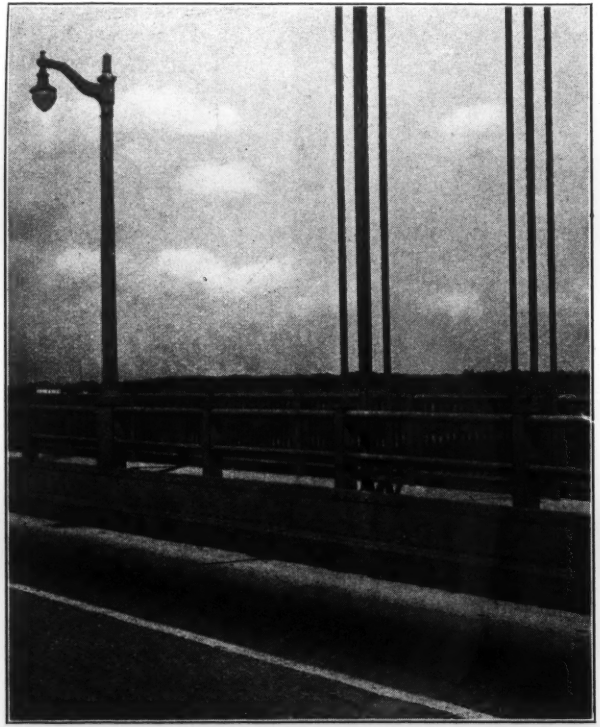


FIG. 25.—VIEW OF DECK, GEORGE WASHINGTON BRIDGE.

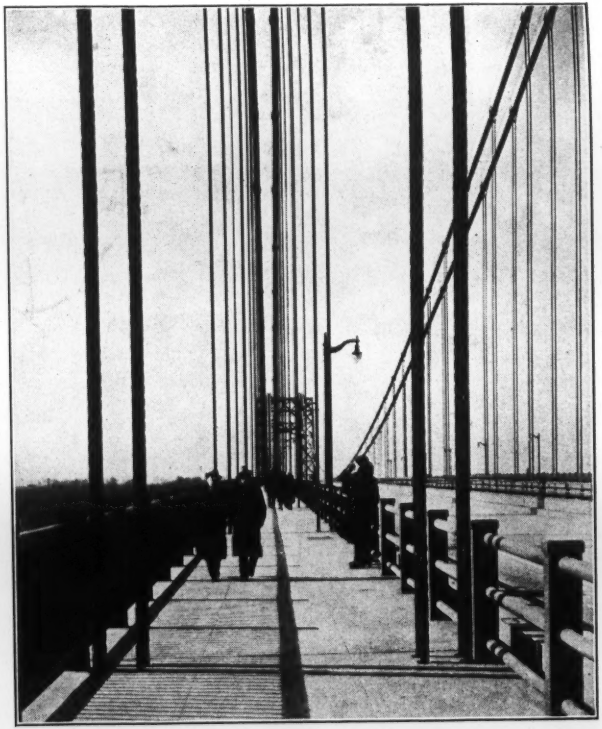


FIG. 26.—VIEW ALONG SIDEWALK, GEORGE WASHINGTON BRIDGE.

of $\frac{3}{8}$-in. material. The longitudinal beams are 14-in., 38-lb. I-beams supported at 20-ft. intervals by angle posts bracketed out from stiffeners on the outside roadway stringer. This was preferred to a direct connection to the ends of the secondary floor-beams, or to the curbs, in order to reduce the transmission of roadway vibration to the sidewalks.

The sidewalks are provided with inner and outer railings connected to the steelwork. (See Fig. 26.) The clear width between railings is 10 ft. 8 in. The suspender ropes, however, pass through the sidewalk slab in two groups of four ropes each, reducing the clear width of the sidewalk between the suspender ropes to 7 ft. 6 in. at each panel point. The posts for the railings are 8-in., 17.5-lb. I-beams, with cast-iron caps, the pipe rails being connected to the webs of the posts with malleable iron fittings. The outer railing consists of top and bottom pipes, $4\frac{1}{2}$ in. and 4 in. in outside diameter, respectively, connected by rectangular steel pickets, $1\frac{3}{4}$ in. by 1 in. in section, welded to the pipes and spaced about 5 in., center to center. The inner railing consists of three lines of pipe, the outside diameter being $4\frac{1}{2}$ in. for the top pipe and $3\frac{1}{2}$ in. for the other two. All the pipes are made of copper-bearing steel. Lighting standards are mounted on the inner railing and are spaced 90 ft. apart, the lights on either side of the roadway being placed opposite each other.

Wind Trusses.—In the initial upper deck construction the wind trusses consist of the upper chords of the future stiffening trusses and diagonals in the plane of these chords. In the completed bridge there will be no wind diagonals in the plane of the lower deck, and the wind load on this deck will be transmitted to the upper deck by the vertical members of the stiffening truss acting with the upper floor-beams as a stiff frame. The lower chords of the stiffening truss, however, will act with the upper chords as chords of the wind trusses, because the diagonals of the stiffening truss will force the lower chord to participate in the stress of the upper chord. Any appreciable relief from such participation, due to the stiffening trusses bowing up and down, is prevented by the resistance of the loaded cable to such motion.

The wind diagonals are made up generally of two carbon-steel angles, 8 by 8 in. by $\frac{5}{8}$ in., placed back to back and held to the under side of the stringers by clamp guides which allow motion between the diagonals and the stringers. The diagonals form an X-system, two panels long, and are connected to alternate floor-beams at the ends and at the center. In the end panels of the side spans and in the four panels at each end of the center span heavier sections of silicon steel are used on account of the greater wind shear in these panels.

The wind chords are box-sections of silicon steel. The upper deck chord, which has an area of 85 sq. in., is made up of two web-plates, 30 in. by $\frac{5}{8}$ in., one top cover-plate, 30 in. by $\frac{1}{2}$ in., two top angles, 6 by 6 in. by $\frac{1}{2}$ in., and two bottom angles, 8 by 8 in. by $1\frac{1}{16}$ in. The horizontal legs of the angles are turned in, and the bottom angles are laced together. As now designed, the lower chords are somewhat smaller, having an area of 75 sq. in. The chords are spliced near the end of each panel for the full value of the member and the ends of the members are milled to bear, no additional provision being made for reversal of stress.

The wind trusses in the center and side spans are entirely separate. The center-span truss has a hinged and sliding end connection to each tower on the center line of the bridge, constructed so as to allow angular and longitudinal motion. The side-span trusses have similar hinged and sliding connections at the towers, but the connections at the anchorages are fixed against both motions, the ends of the chords being connected to frames embedded in the anchorage masonry which take the longitudinal reaction, while the wind shear reaction is taken by the anchorage floor-beams.

Analysis of Wind System.—The wind loads acting on the cables and the floor structure are transmitted to the towers and anchorages partly by the cables and partly by the wind trusses. These two carrying members, however, are interdependent since they are connected by the suspenders. In the case of the long center span the wind truss is much more flexible than the cables. Its greater deflection from wind load moves the suspenders from their vertical plane, the lateral component of the suspender pull tending to restrain the deflection of the truss and, at the same time, to increase the lateral deflection of the cables. This restraining effect is particularly great near the middle of the span, where the suspenders are short, and where the lateral component of the suspender pull is greater than the wind load on the floor.

The wind truss, therefore, has not only end reactions at the towers, but reactions all along its length, from the suspenders, which are small near the ends and large near the center. The maximum moment occurs near the quarter-points.

In the case of the relatively short side spans the deflections of the cables and the wind trusses are small, and nearly equal, so that the effect of the suspenders is negligible, and the wind load on the floor was assumed to be carried by the wind truss alone.

To find the moments and shears in the center-span wind truss it was necessary to determine the amount of the lateral suspender pull or reaction at every point along the truss. This was done by a "cut-and-try" method of assuming the lateral suspender pulls, finding the deflections of the cables, suspenders, and truss, and then revising the assumed pulls until these deflections were in agreement at all points; in other words until the deflection of the truss at every point was equal to the sum of the deflections of the cables and suspenders at that point, or (referring to Fig. 27), $d_t = d_c + d_s$.

The deflections of the truss were found by the moment-area method, corrected for the effect of the diagonals, with the floor wind loads acting in one direction and the assumed lateral suspender pulls acting in the opposite direction, or (referring to Fig. 27), under loads of $w_f - q$. The deflections of the cables were found by determining the equilibrium polygon for the wind loads on the cables and the assumed lateral suspender pulls combined, or (referring to Fig. 27), $w_c + q$, with the horizontal component of the cable pull equal to that due to the vertical loads. The deflections of the suspenders were found by the principle that the deflection of a suspender bears the same relation to its length as the lateral component of its pull bears to its pull, or (referring to Fig. 27), $d_s : h = q : p$ In Fig. 27, d_s, d_c, and d_t are the lateral

deflections of the suspender, cable, and wind truss, respectively; w_c and w_f are wind loads on cable and floor; h is the length of suspender; p is the suspender pull; and q is the lateral component of suspender pull.

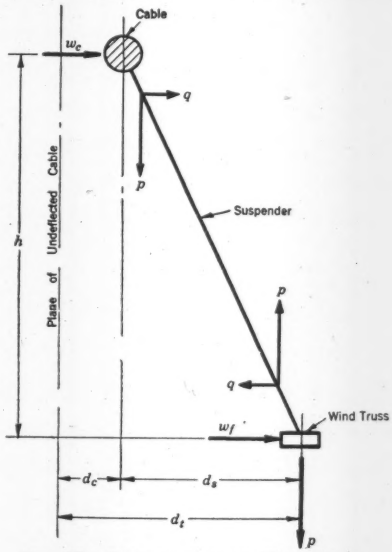


FIG. 27.—DISTRIBUTION OF WIND LOAD.

The truss deflections, and the cable deflections with the suspender deflections added, were plotted and the two curves compared. The assumed lateral suspender pulls were then corrected, and the deflections re-calculated until the curves were brought into agreement.

The lateral suspender pulls may be considered as transferring floor wind load to the cables. This distribution of the wind load between the cables and truss is shown in Fig. 28, which also shows the deflection curves and moment and shear diagrams. In this case, which is for the completed bridge, 65% of the floor wind load is transferred to the cables. The maximum truss deflection which occurs at the center is 11.8 ft. A deflection of this magnitude, while not objectionable, is not likely to occur because a steady wind pressure of 30 lb. per sq. ft. over the entire span is highly improbable. To reduce this deflection materially by increasing the chord areas would have been expensive. Even doubling the chord areas would have reduced the deflection only to 10.6 ft.

Deflections and stresses were also determined for the initial upper deck construction; but, as the reduction in wind pressure is greater than the reduction in weight of the structure, the deflections and stresses were found to be smaller than for the completed bridge, the maximum deflection being 10.7 ft.

Stiffening Trusses.—The future stiffening trusses as now designed have a depth of 29 ft. center to center of chords and are Warren trusses with two

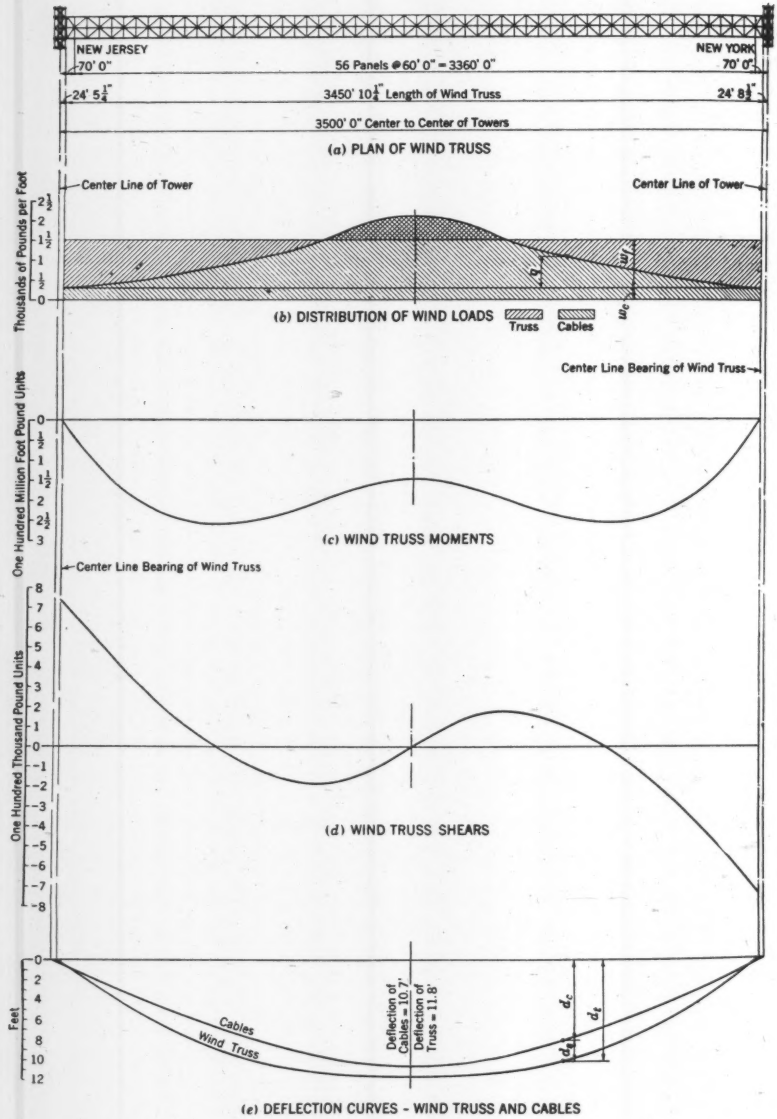

NEW JERSEY
70' 0"
56 Panels @ 60' 0" = 3360' 0"
NEW YORK
70' 0"
24' 5 1/4"
3450' 10 1/4" Length of Wind Truss
24' 8 3/4"
3500' 0" Center to Center of Towers
(a) PLAN OF WIND TRUSS
Thousands of Pounds per Foot
Center Line of Tower
Center Line of Tower
(b) DISTRIBUTION OF WIND LOADS
Truss
Cables
Center Line Bearing of Wind Truss
One Hundred Million Foot Pound Units
(c) WIND TRUSS MOMENTS
Center Line Bearing of Wind Truss
One Hundred Thousand Pound Units
(d) WIND TRUSS SHEARS
Feet
Cables
Wind Truss
Deflection of Cables = 10.7'
Deflection of Truss = 11.8'
(e) DEFLECTION CURVES - WIND TRUSS AND CABLES

FIG. 28.—WIND SYSTEM, CENTER SPAN.

diagonals in each 60-ft. panel. The make-up of the cross-section of the upper chords, which have an area of 85 sq. in. and are of silicon steel, has previously been given in the description of the wind trusses. The lower chords, which have an area of 75 sq. in., are box sections of silicon steel consisting of two web-plates 24 in by $\frac{5}{8}$ in., one top cover-plate, 30 in. by $\frac{1}{2}$ in., two top angles, 6 by 6 in. by $\frac{1}{2}$ in., and two bottom angles, 8 by 6 in. by $\frac{11}{16}$ in. The horizontal legs of the angles are turned in, the long legs of the bottom angles being horizontal and laced together. The diagonals are also box sections, the typical diagonals being of carbon steel and having an area of 39 sq. in. They are made up of two web-plates, 20 in. by $\frac{1}{2}$ in., and four angles, 6 by 4 in. by $\frac{1}{2}$ in., with the 6-in. legs turned in and laced together. Seven diagonals at each end of the center span, three diagonals at the tower ends of the side spans, and four diagonals at the anchorage ends of the side spans, require stronger sections than the typical sections, and these are obtained either by the use of silicon steel, or by heavier sections, or both. Gusset-plates are now provided on the upper chords for the connection of these future diagonals, but are left blank, the holes to be drilled in the field. The center-span stiffening trusses are supported at the towers by pinned hangers connecting the end panel points of the bottom chords to brackets on the towers, as shown subsequently under "Details at Tower," in Fig. 30. Similar hangers support the side-span trusses at the towers, and the shore ends are connected to the frames embedded in the anchorage masonry by the pins now connecting the top chord.

The stiffening trusses are very flexible, having a ratio of depth to span of 1:120 in the case of the center span, and 1:20 and 1:18, respectively, for the New York and New Jersey side spans. As a result they have no appreciable effect upon the deformation of the cables, except for very short and heavy loads, and, even then, their effect is slight. Since the deformation of the cables is so little affected by the truss stiffness and since the trusses are forced to take the same deformation, it follows that the live load unit stresses in the truss chords are practically independent of their cross-sectional area and vary in almost direct proportion to the depth of the truss. The depth used for the trusses is the minimum permitted for the overhead clearance required for the lower deck and the detail arrangement of the floor system. This depth practically determined the live load unit stresses in the chords, and since the unit stresses from wind (as has previously been explained), are only slightly affected by the chord areas, the proportioning of the chords became simply a matter of selecting minimum practical cross-sections and choosing a grade of steel capable of withstanding the imposed unit stresses.

As previously stated, the trusses are designed for a partial live load of 23 000 lb. per ft. of bridge, reduced for length of load and number of loaded lanes, together with the impact percentage as given by the specifications; and, in addition, an extended load of 4 000 lb. per ft. over the entire structure, including the section covered by the partial load. However, it was found in all cases that this load of 4 000 lb. per ft. actually decreased the stresses from the partial load by a very small amount, and it was not used at all in determining the stresses. The lengths of load for the partial live

load were taken as multiples of the panel length. By trying various lengths it was found that a loaded length of six panels, or 360 ft., gave maximum chord stresses for the entire center span. For this length the reduced partial load is 7 700 lb. per ft. of bridge and the impact, 10 per cent. For maximum shears in the center span, it was found that the loaded length should be four panels, or 240 ft., the reduced partial live load for this length being 8 900 lb. per ft., with an impact of 13 per cent. For the side spans it was found that a partial live load covering the entire span gave maximum chord stresses, the reduced partial live load being 6 700 lb. per ft., with an impact of 7 per cent. The load positions for maximum shears in the side spans were the same as those for a simple span.

In the center span the maximum unit stresses from live load and impact are 15 800 lb. per sq. in. compression on the gross area of the upper chord, and 22 500 lb. per sq. in. tension on the net area of the lower chord. These maximum stresses occur near the ends of the span. The maximum unit stresses from wind are 15 500 lb. per sq. in. compression in the upper chord and 19 100 lb. per sq. in. tension in the lower chord. These maximum stresses occur near the quarter-points of the span. By combining the maximum live load and impact stresses with one-half the wind stresses at the same point in the truss, or the maximum wind stresses with one-half the corresponding live load and impact stresses, as required by the specifications, the total stresses (including the small stresses due to temperature) were found to come within the specified allowable unit stresses for silicon steel for such load combinations. These specified stresses are: A compression of 24 400 lb. per sq. in. for the upper chord $\left(\frac{l}{r} = 60.5\right)$; and a tension of 29 700 lb. per sq. in. for the lower chord.

In the side spans the unit stresses from live load and impact are somewhat less than in the center span, and the wind stresses very much less, so that the total stresses are well within the allowable unit stresses for silicon steel, although greater than those for carbon steel.

In the final calculations for the live load moments and shears in the stiffening trusses, the formulas of the so-called "exact" or "deflection" theory were used for most of the center span. For the side spans, however, and for short sections at each end of the center span, these formulas give incorrect results because they neglect the stretch of the long suspenders and because the supports for the cables and trusses are not in the same vertical line. For the side spans and for the ends of the center span, therefore, a "cut-and-try" method was used, which consisted in assuming the distribution of live and dead load between the truss and cables, and then computing the deflections of the truss and the deflections of the cables, including the stretch of the suspenders. The assumed load distribution was then revised until these two sets of deflections were in agreement.

Fig. 29 gives the curves of maximum moments and shears for the center span and also the curves of loading, shear, moment, and deflection for a typical load case.

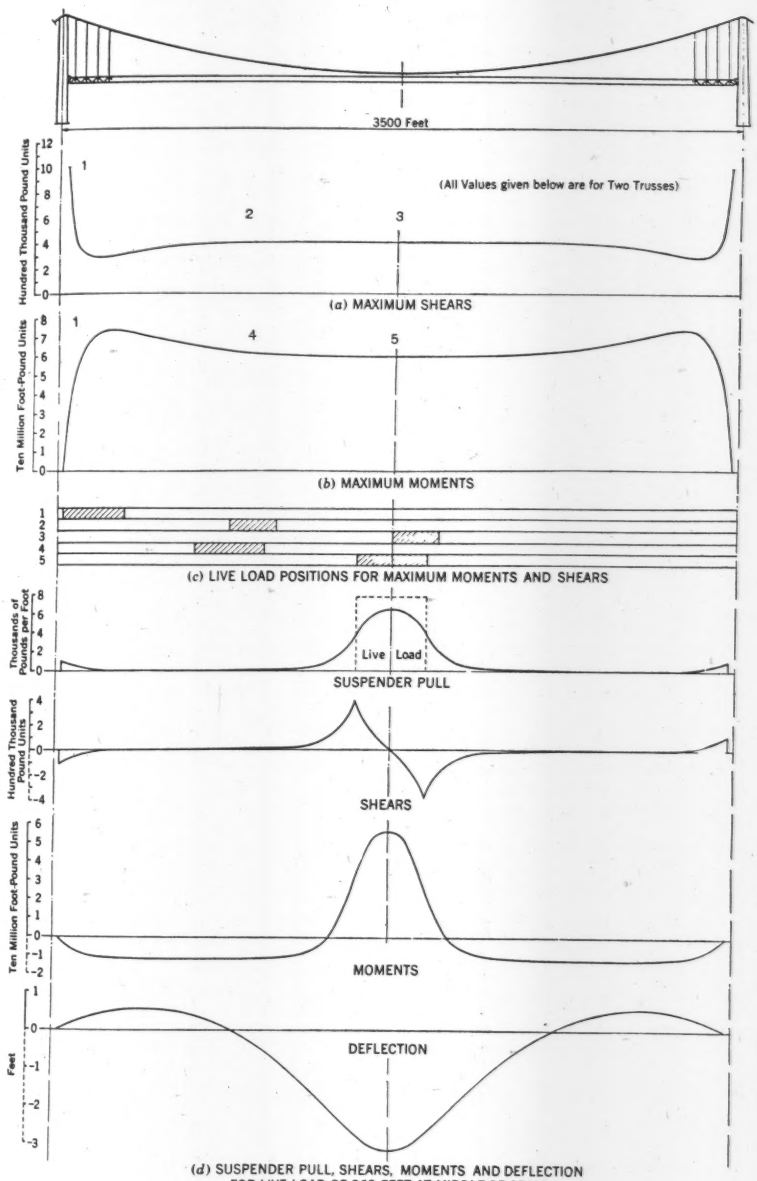


FIG. 29.—STIFFENING TRUSSES, CENTER SPAN.

Details at Towers.—The connections that transmit vertical, transverse, and longitudinal reactions from the suspended floor structure to the towers are designed to permit unrestricted angular movement in both horizontal and vertical planes as well as longitudinal movement. Figs. 30, 31, and 32 show the details at the towers for both the center and the side spans.

At each tower the steelwork of the roadway is supported on the transverse struts of the tower. The stringers which span between the tower and the first suspended floor-beam are pin-connected to the floor-beam, and have sliding bearings on the tower strut. The wind chords in this panel are connected at the floor-beam by horizontal pins to the wind chords of the adjacent panel, and the ends at the tower are supported by brackets from the outside stringers. The wind laterals in this panel consist of (1) diagonals from the ends of the chords to the center of the floor-beam, where a transverse shear connection and a horizontal pin-joint are provided; and (2) transverse struts from the ends of the chords to a connection with the tower at the center line of the bridge. This connection consists of a tongue, 5 ft. wide, made up of plates and stiffener angles, which is free to move longitudinally over the top of the tower strut between cast-steel guides. The outer faces of these guide castings are cylindrical and bear against the concave faces of castings fixed to the tower strut, thus allowing rotation as well as sliding of the tongue, the wind shear being transmitted to the tower by these castings. The longitudinal motion of the tongue at each end of the center span is limited by a bumper fastened to a longitudinal tower strut, the contact surface between the tongue and the bumper being curved to allow for rotation.

The clearance between the tongue and the bumper is made large enough to allow some longitudinal movement of the entire suspended floor structure, even at the highest temperature. Such longitudinal movement may be caused by longitudinal wind and breaking forces, these forces amounting to a total of 3 500 000 lb. If resisted by the suspenders and cables alone this force would move the structure 24 in. The bumpers were introduced in order to reduce the length of the expansion details and, in limiting the motion, they take part of the longitudinal force. At the highest temperature the movement permitted by the bumpers is $4\frac{3}{4}$ in., and the bumper reaction is 2 800 000 lb. This force is transmitted to the bumper by a longitudinal strut between the tongue and the intersection of the diagonals at the floor-beam. This strut is braced to the bottom flanges of the adjacent stringers to form a wide trussed member which carries the moment caused by the eccentricity of the transverse wind reaction.

The expansion joint in the roadway slab on the center-span side of the tower is formed by two wide gratings, one attached to the floor steel of the tower and the other attached to the floor steel of the suspended structure. These gratings resemble combs in form, with their teeth intermeshing over a common support upon which the teeth from the suspended structure slide. (See Fig. 33.) The teeth are rolled steel flats, 56 in. long and $\frac{5}{8}$ in. thick, welded together in 2-ft. sections, with $1\frac{1}{16}$-in. spacers between them. They are 5 in. deep on the moving side and $6\frac{1}{4}$ in. deep on the fixed side, and their top surfaces are roughened by transverse grooves.

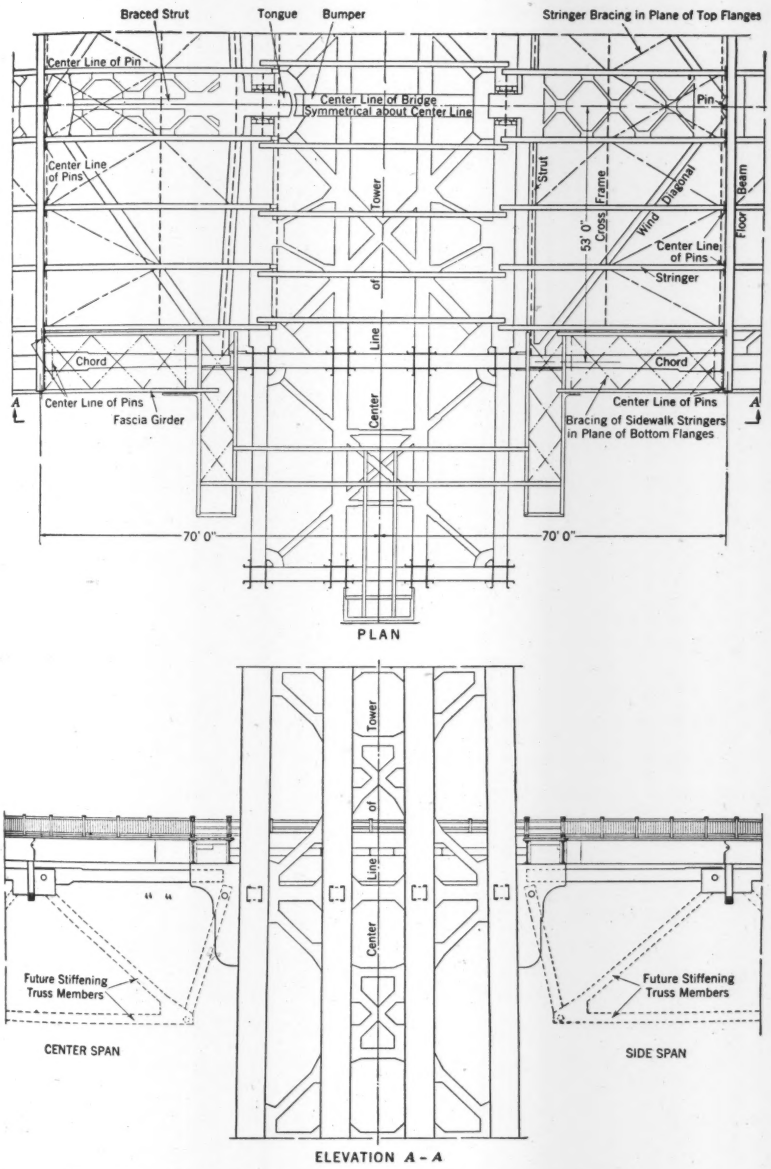
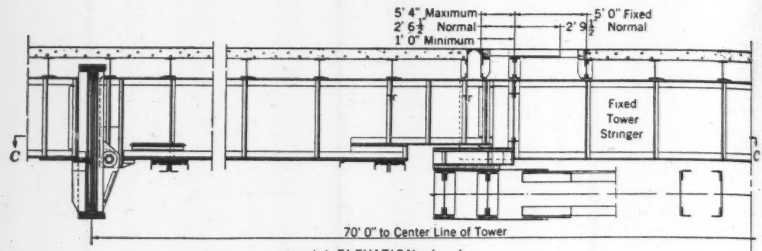
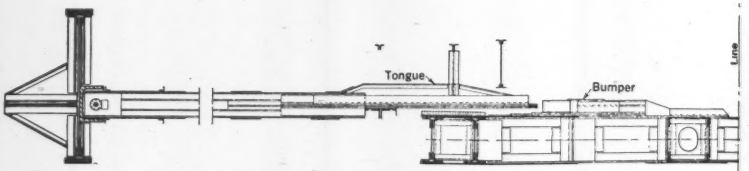


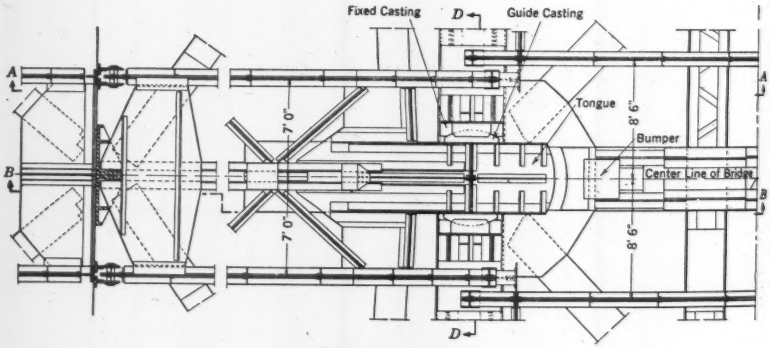
FIG. 30.—FLOOR FRAMING AT TOWER, GENERAL PLAN AND ELEVATION.



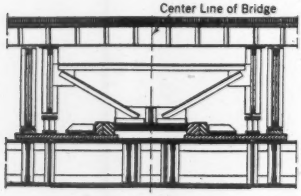
(a) ELEVATION *A - A*



(b) ELEVATION *B - B* OF TONGUE AND BUMPER



(c) SECTIONAL PLAN *C - C*



(d) SECTION *D - D*

FIG. 31.—FLOOR EXPANSION DETAILS AT TOWER, CENTER SPAN.

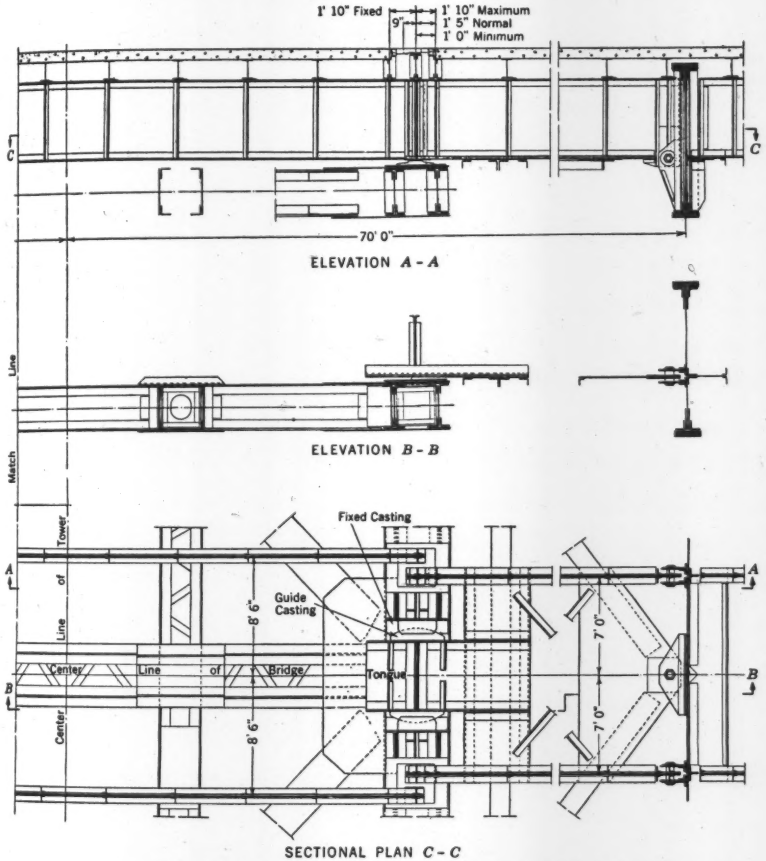


FIG. 32.—FLOOR EXPANSION DETAILS AT TOWER, SIDE SPAN.

The temperature range of 110° produces the following range of movement, in inches, in the center-span gratings at each tower:

Change in length of floor	$= +15$
Deflection of tower at grating level	$= +\frac{5}{8}$
Change in length of floor due to change in camber	$= +\frac{5}{8}$
Angular tilt of end of stiffening truss	$= -1\frac{1}{4}$
Total	$= +14\frac{1}{2}$

The variation in bumper clearance due to temperature is the same as that in the grating, with the exception of the effect of the angular tilt of the stiffening truss which is $\frac{3}{4}$ in. less at the bumper level. Thus, the total change in bumper clearance due to temperature is $15\frac{1}{4}$ in., and it varies from a minimum of $4\frac{3}{4}$ in. at + 105° to a maximum of 20 in. at — 5 degrees.

The combined effect of temperature variation and longitudinal forces produces the following range of movement, in inches, in the roadway grating:

Temperature range	$14\frac{1}{2}$
Movement permitted by bumper at highest temperature	$4\frac{3}{4}$
Movement permitted by bumper at lowest temperature	20
Total	$39\frac{1}{4}$

The angular deflection from transverse wind load results in a longitudinal motion at the curb line of $7\frac{1}{2}$ in. For the maximum movement at lowest temperature, however, a 45° wind is assumed, since its longitudinal com-

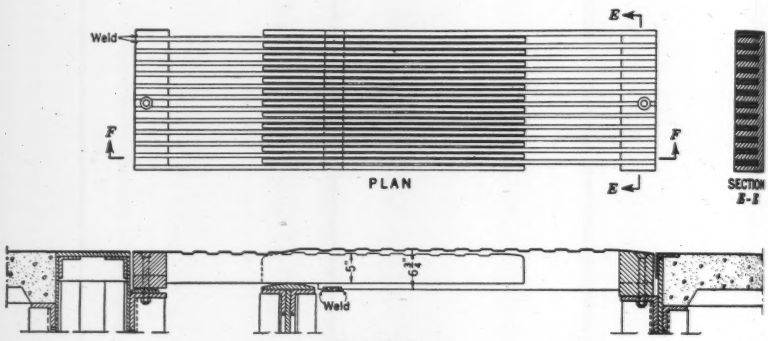


FIG. 33.—DETAIL OF GRATING, CENTER SPAN.

ponent is necessary to produce the drift of 20 in., and the movement at the curb line due to its transverse component is 5 in. For the maximum movement at highest temperature, the drift of $4\frac{3}{4}$ in. may be produced by the breaking force alone, so the wind movement at the curb line for this case may be the full $7\frac{1}{2}$ in. Thus, due to wind, the range of movement at the curb line is $12\frac{1}{2}$ in. greater than at the center line, giving a range of movement in the grating at the curb line due to temperature, longitudinal forces, and wind, of $51\frac{3}{4}$ in., of which, $19\frac{1}{2}$ in. is a closing from normal and $32\frac{1}{4}$ in. is an opening from normal.

A partial live load at one end of the span causes an angular tilt of the stiffening truss outward from the tower at the loaded end, which causes an opening at the grating of $3\frac{3}{8}$ in. However, at the unloaded end of the truss, there is an inward angular tilt which reduces the possible longitudinal movement permitted by the bumper by $\frac{3}{8}$ in., so that the net opening from live load is 3 in. The simultaneous occurrence of this partial live load with maximum wind, temperature, and longitudinal force, is so improbable that it was not considered necessary to allow for a combination of all these movements in the expansion joint, and the grating was designed to allow a range of movement of 52 in., of which $18\frac{1}{2}$ in. is a closing from normal and $33\frac{1}{2}$ in. is an opening from normal.

For the side spans the connections to the tower are similar to those for the center span, but the motions provided for in the expansion grating are much smaller, having a total range of only 10 in. Since longitudinal forces on the side spans are taken into the anchorages by embedded frames, no bumper is provided at the tower, and there is no need to allow for movement from longitudinal forces in the grating.

The expansion joints in the sidewalks are formed by sliding plates allowing a range of movement of 62 in. for the center span, and 16 in. for the side spans.

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Paper No. 1821

GEORGE WASHINGTON BRIDGE: DESIGN OF THE TOWERS

BY LEON S. MOISSEIFF,[1] M. AM. SOC. C. E.

SYNOPSIS

The unusual type of tower adopted in the design of the George Washington Bridge is discussed in this paper. The entire load of the bridge is applied at the top of the inner columns of the tower and is distributed between all columns as the stress proceeds downward, until finally the reaction on the masonry is practically uniform. The paper demonstrates how this tower was analyzed; methods used to test, analytically, the efficiency of the design are related; stress-strain measurements on a celluloid model of a tower bent are described; and results are given of stress-strain measurements on the completed towers, which show the distribution of the load in the tower columns. These final measurements demonstrate the agreement between engineering theory and the actual behavior of the structure. Tests of large sized columns were made in conjunction with the National Bureau of Standards, and their results are reported. Conclusions are drawn as to the efficiency and safety of structures of this magnitude.

INTRODUCTION

By their position in the landscape, their rising mass, and the emphasis which the ascending curves of the cables throw to them, the towers of a suspension bridge exert a preponderant influence on the æsthetic impression of the observer. When the bridge is viewed as a whole they at once attract the eye and concentrate the attention. Of the three principal parts of a suspension bridge—the anchorages, cables, and towers—the anchorages are too far away and are too often hidden by the design or by other structures to be felt as part of the picture, while the cables by their continuity and rela-

[1] Advisory Engr. on Design, The Port of New York Authority; Cons. Engr., New York, N. Y.

tively small size do not present a restful object. As a matter of fact, the towers of a suspension bridge not only sustain the entire weight of the superstructure, but also the upward pull of the cables at the anchorages, and the greatest forces are concentrated on them. The statical importance of the towers is justly felt by the public at sight, and they, therefore, demand the best æsthetic treatment.

Large public works have their history and origin that influence their development and final form. It requires time for the demand for a public bridge involving large expenditures and engineering difficulties to grow and to become strong enough to be realized. In the course of time the need for traffic facilities grows and the demand for them becomes more pressing. At the same time, the available funds, the engineering knowledge, the technical capacities of the mill, the fabricator, and the erector, also grow. During the period in which the idea of the bridge is being "incubated," the engineering phases of the structure take form through public opinion and engineering study, which mature later into the completed structure.

The story of the growth of the idea for bridging the Hudson River, which has finally culminated in the George Washington Bridge, has been completely told in the paper[2] by O. H. Ammann, M. Am. Soc. C. E. It began in the last ten years of the Nineteenth Century and continued through forty years. The great advances made during this period in the theory of structures, the knowledge of materials, the making of steel, the fabrication and erection of bridges, all find themselves reflected and embodied in the bridge as built; but so also do many notions and traditions of older times. This especially applies to the towers of the George Washington Bridge.

The age of metal structures is still young; that of stone goes back to time immemorial. For many centuries, habit, which determines æsthetic feeling to the largest extent, saw beauty in the massiveness of stone only. The shadow of the well-proportioned masonry towers of the old Brooklyn Bridge fell upon the proposed towers of the George Washington Bridge.

The genetics of the bridge project over the Hudson River were such that, conforming to public opinion and its habitual taste, the bridge was first pictured with massive towers. When after years the project was finally approaching realization, the towers were built as steel structures of strength and stability to sustain all the loads that may come on the bridge, and provisions were made in their design to allow for embedding or enclosing them in concrete masonry so as to obtain the effect of a monumental structure. As a matter of fact, the appearance of the bare steel towers is better than was expected, and they seem to have gained the favor of the general public. The writer is of the opinion that they will probably remain without enclosure, as shown in Fig. 1.

The fact that the towers were conceived to be ultimately encased in masonry is important to the understanding of their design and their articulation. It determined the main features and the novelty of proportioning. The problem to be met was how best to design steel towers for the required dimensions and forces.

[2] See p. 1.

As shown in Fig. 2(*b*) each tower is composed of four rigidly connected upright frames. Each frame consists of two inner and two outer columns (Fig. 2(*a*)) with bracing between to connect them, making a total of sixteen columns. The inner eight columns are vertical in transverse elevation and the outer eight columns are inclined. The two sets are connected by a double bracing of diagonal and horizontal members in twelve panels as shown. The two halves of each frame are connected by three braces, one below the roadway and the other two close together at the top. Longitudinally, the four frames are held together by a double bracing of diagonals and horizontals in the planes of the inner and outer columns as well as in several planes of the braces to assure unity of action and equal distribution under load.

The steel towers are proportioned to sustain the entire load ultimately imposed on them. To reach a conception of these forces the tremendous size of the George Washington Bridge should be fully realized. The towers must sustain the weight of the entire suspended structure—amounting to 39 000 lb. per ft. in the center span, 41 400 lb. per ft. in the side spans—and the live load on the bridge, as well as the vertical uplift on the anchorages. These weights produce a vertical reaction on top of each tower of 112 000 tons. In addition, each tower must sustain its own weight of 20 000 tons and almost the full wind pressure on the structure.

The saddles transmitting the entire suspended weight of the bridge to the towers are centrally located above the inner columns. This makes it impossible to engage all members of the tower to their full efficiency. Part of the steel is required to transmit and distribute the stress to all sixteen columns.

This condition could have been avoided by transferring one-half the load to the outer columns directly by placing the saddles and cables midway between the inner and outer columns. The arrangement would have increased the distance between center lines of cables from 106 ft. to 144 ft. It would have caused an equivalent increase in the length of all floor-beams, resulting in an additional 12 000 tons of steel. Adding to it the increased wire required to sustain this additional weight and estimating the total increase at contract prices, an extra cost of $2 800 000 would have been incurred. The effect on the anchorages has been omitted in this estimate; it would add still more to the cost. On the other hand, to locate the saddles and cables concentrically would have saved less than $100 000 in the steel of the towers. The advantage of this arrangement would be the central location of the cable reaction in the tower bent and concentric distribution of stress that is assumed to follow from it. Its disadvantage is this very considerable additional cost. The more economical design was adopted.

Analysis of Steel Towers

In the computations and the proportioning of the members, the four frames forming the tower were assumed to participate equally in sustaining all vertical loads. Therefore, one frame only was analyzed for loading conditions, taking one-quarter of the total load. Each unit forms a three-story frame with double diagonal bracing between the columns. The frame is statically

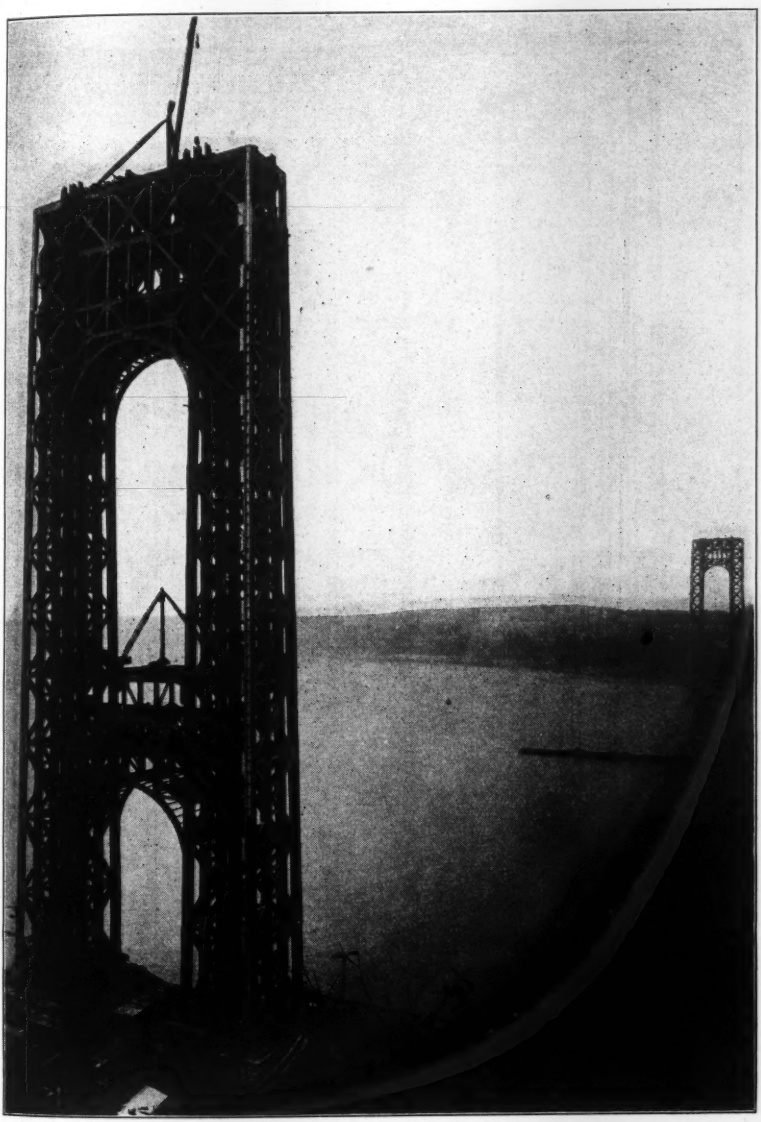


FIG. 1.—VIEW OF NEW JERSEY TOWER DURING CONSTRUCTION.

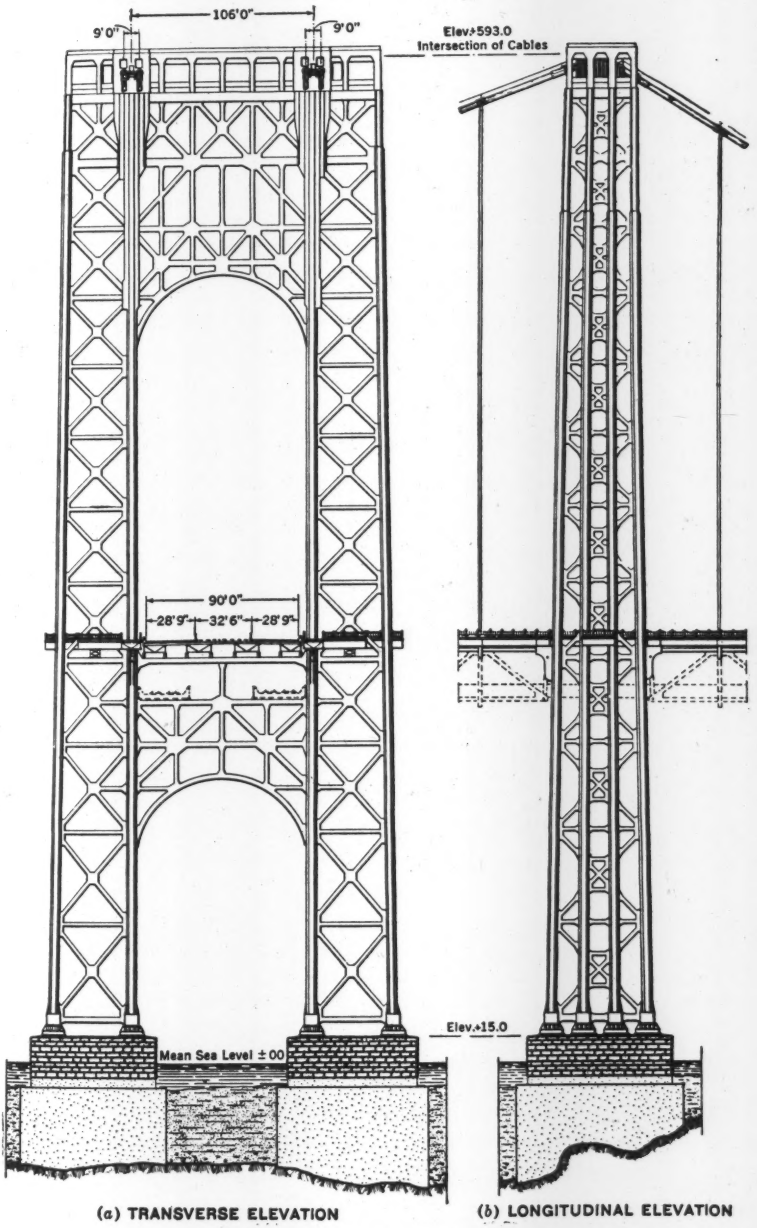


FIG. 2.—GENERAL DRAWING OF THE TOWER.

indeterminate to a high degree, but its elastic behavior is easily visualized. The main external loads are the cable reactions applied at the center of the inner columns. These loads compress the inner columns and tend to shorten them. The braces prevent the columns from bending and part of the stress is forced through the diagonals to the outer columns.

To facilitate the computation of the stresses, the frame was first divided into two superimposed parts, each containing a single diagonal system between the columns. The frame was then transformed into a statically determinate system by passing a vertical plane along the center line, and each member cut by this plane was replaced by an unknown force. This, however, was not quite sufficient to transform the frame into a statically determinate system, some additional redundant members of the braces had to be cut, thus increasing the number of unknown forces to be determined by the applications of the elastic theory. As seen in Fig. 3, a total of sixteen unknowns is required

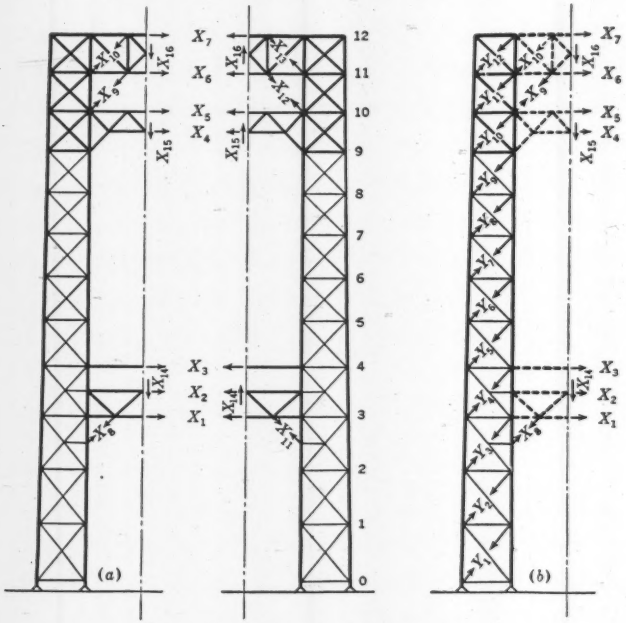


FIG. 3.—FORCE DIAGRAM; DESIGN OF TOWERS.

to make the structure statically determinate for any loading case. Each half of the frame acts as a vertical cantilever, subject to the external and unknown forces. It will be readily seen that for symmetrical vertical loads the forces, X_{14}, X_{15}, and X_{16}, will be zero, and that also Force X_{11} = Force X_8; Force X_{12} = Force X_9; and Force X_{13} = Force X_{10}. For this case of loading, therefore, the number of unknown forces is reduced to ten.

Horizontal forces acting at corresponding points on each side of the center line in the same direction and of equal magnitude will result in the forces, X_1, X_2, X_3, X_4, X_5, X_6, and X_7, being equal to zero, and Force X_8 = Force X_{11}; Force X_9 = Force X_{12}; and Force X_{10} = Force X_{13}, thus reducing the number of unknowns to six.

Following this procedure the elastic theory furnishes the number of equations necessary to compute the numerical values of the unknown forces for any loading. These are then treated as external loads acting on a statically determinate system. The application of this theory is well known. At the outset it requires the assumption of the sectional areas of all members of the tower. The computations are repeated several times, therefore, until a close agreement is reached between the assumptions made and the results obtained.

The analysis and the procedure for establishing the equations was along the following lines: The stresses in the system with all redundant members cut as indicated in Fig. 3 were denoted as u and p stresses. The u stresses are those due to the action of the redundant members, X_1, X_2, etc., so that u_1 is due to $X_1 = -1$; u_2 is due to $X_2 = -1$; ... u_{16} is due to $X_{16} = -1$. The p stresses are due to the external loading, considering X_1, X_2 ... X_{16} equal to zero.

The u and p stresses for the tower columns are the average of the stresses of two systems, each with a single set of diagonals. If v is the stress due to $X = -1$ for diagonals sloping downward toward the inner column, and w is the stress due to $X = -1$ with diagonals sloping downward toward the outer columns, $u = \frac{1}{2}(v + w)$.

The unknowns are evaluated by the solution of the simultaneous elastic equations which read as follows:

$$X_1 \Sigma \frac{u_1^2 L}{EA} + X_2 \Sigma \frac{u_1 u_2 L}{EA} + X_3 \Sigma \frac{u_1 u_3 L}{EA} + \ldots X_{16} \Sigma \frac{u_1 u_{16} L}{EA} = \Sigma \frac{p u_1 L}{EA} \quad ..(1)$$

$$X_1 \Sigma \frac{u_2 u_1 L}{EA} + X_2 \Sigma \frac{u_2^2 L}{EA} + X_3 \Sigma \frac{u_2 u_3 L}{EA} + \ldots X_{16} \Sigma \frac{u_2 u_{16} L}{EA} = \Sigma \frac{p u_2 L}{EA} \quad ..(2)$$

.....

$$X_1 \Sigma \frac{u_{16} u_1 L}{EA} + X_2 \Sigma \frac{u_{16} u_2 L}{EA} + X_3 \Sigma \frac{u_{16} u_3 L}{EA} + \ldots X_{16} \Sigma \frac{u^2_{16} L}{EA} = \Sigma \frac{p u_{16} L}{EA} \quad ..(16)$$

in which, Equations (3) to (15), inclusive, can be interpolated and in which, L = length of members; A = area of members; and E = modulus of elasticity, which being constant eliminates itself.

The stress in any member of the bracing between the bents is equal to,

$$s = p - X_1 u_1 - X_2 u_2 - X_3 u_3 \ldots X_{16} u_{16} \ldots\ldots\ldots\ldots (17)$$

The fact that two systems—each with a single set of diagonals between the columns—have been superimposed neglects the effect of participation stresses caused by the double diagonal bracing. A vertical load on the towers causes shortening in the columns and thereby compresses the diagonals attached to them. Such stresses, of an inductive nature, are called participation stresses. These stresses are purely of local character and do not in any way

affect the unknowns previously mentioned and, hence, the relative distribution of stresses in the columns remains the same. The participation stresses affect mostly the diagonals and the horizontals, the effect on the column stresses being very slight. The stresses in the diagonals could be approximated very closely in a simple manner for each panel independently after the column stresses have been evaluated. This was done in fact in the earlier computations until close agreement was reached between the assumed and required areas. In the final computations, however, one set of diagonals was introduced as unknowns and their values were found, together with the stresses in the other members of the bent, by application of the elastic theory similar to the first part of the analysis. The previously computed unknowns were introduced as external forces. The arrangement of the unknowns for participation stresses is shown in Fig. 3(*b*). The analysis was as follows: Denote by $\bar{u}_1$ stress due to $Y_1 = -1$; $\bar{u}_2$ stress due to $Y_2 = -1$; ... $\bar{u}_{12}$ stress due to $Y_{12} = -1$.

Let $\bar{p}$ denote stress due to external loading plus stress due to the forces, X, as previously found, so that:

$$\bar{p} = p' - X_1 v_1 - X_2 v_2 \ldots X_{16} v_{16}$$

in which, p' is the stress due to the external loading in the statically determinate system as shown in Fig. 3(*b*). After solving the elastic equations, the stress becomes:

$$s = \bar{p} - Y_1 \bar{u}_1 - Y_2 \bar{u}_2 \ldots Y_{12} \bar{u}_{12}$$

It is assumed also in this more refined analysis that the horizontal shear between the inner and the outer columns is equally sustained by both members of the double diagonal bracing. To eliminate this assumption all unknowns of both systems should be introduced simultaneously, but this was not considered necessary.

The loading case of most importance is the vertical cable reaction acting on the center line of the inner columns. The three braces between the bents keep the columns in vertical position. The tendency of the bents would be to deflect inward at the top, but this is prevented by the top brace. They tend to curve outward, therefore, through their height and cause tension in the middle and lower brace. The compressive force in the top brace amounts to 5.8% of the total cable reaction as compared to a tension in the middle and lower brace of 3.9% and 1.3%, respectively.

The reaction of the saddles on the inner columns is distributed gradually by their elastic shortening through the diagonals to the outer columns. The load thus transmitted is small at the top and gradually increases, toward the bottom. The outer columns in the top panel sustain only 4.3% of the reaction, which increases to 37.8% in Panel 5-6 and reaches 46.5% at the bottom. The diagonals in the upper panels are most effective in transferring stress to the outer columns.

Longitudinal Tower System

The stretching and contracting of the cables in the side spans caused by live load and temperature changes will bend the towers and produce stresses

in them. It is evident that the towers will follow the tremendous forces of the cables and that their tops will be pulled into a position such that the equilibrium of forces is again established. This will be resisted by the posts as well as the diagonals in both longitudinal and transverse planes. The tower will act as a unit, each frame contributing in proportion to its distance from the neutral axis. Two distinct steps are required to analyze the stresses caused by the bending of the towers. One is to find the horizontal force at the tops of the inner columns which, together with the vertical cable reaction and the tower weight, will force the structure to deflect in accordance with the cable movements; and the second is to compute the stresses in the towers, which act in this condition as vertical cantilevers.

Longitudinal wind forces acting directly on the towers and wind forces transmitted from the suspended structure will also tend to deflect the towers. For this case, however, the towers are held in place by the cables and act as vertical beams fixed at the base and hinged at the top.

In computing the bending stresses due to longitudinal bending and wind, the two inner frames were assumed to be replaced by similar structures acting in the plane of the outer frames. The sections of the inner frames were reduced according to their distances from the center of gravity of the acting masses. The diagonals in the plane of the frames were also taken into account in resisting distortion.

Each tower actually consists of four vertical trusses in longitudinal planes. These trusses are tied together by four horizontal braces at Panel Points 4, 10, 11, and 12. The horizontal force at the top in the case of tower bending, the reaction of the saddles, and the transferred wind at floor level in the case of longitudinal wind pressure are all applied directly on the two trusses formed by the inner columns only. The two remaining trusses formed by the outer columns are forced into action by the horizontal braces. If the tower is analyzed for symmetrical, longitudinal loads applied at the saddles, it is four times statically indeterminate for bending, the shears transferred through the four horizontal braces from the inner to the outer columns being the unknowns. For longitudinal wind pressure on the tower the unknowns are increased to five, the fifth being the wind reaction at the top.

The horizontal braces have been found to be very efficient. The stress transferred to the outer columns by bending is equal to 43% in Panel 5-6 and increases to 47% at the base; the wind stresses are equally well distributed.

Studies of Efficiency of Tower Design

The cable concentrations on the inner columns presented a novel feature in the design of bridge towers. The efficiency of this design in distributing the stresses through its members to the piers had to be established beyond all reasonable and possible doubt. In developing the design of the towers it was fully realized that the mathematical analysis and the proportioning of the various members are based on tacit assumptions that pre-suppose the unison of the behavior of the constituent members, and especially the continuous

elastic action of the frame. The basis of the design also assumes that the modulus of elasticity of each of the steel members is practically the same in all parts, which assumption underlies the design of all engineering structures for the same materials.

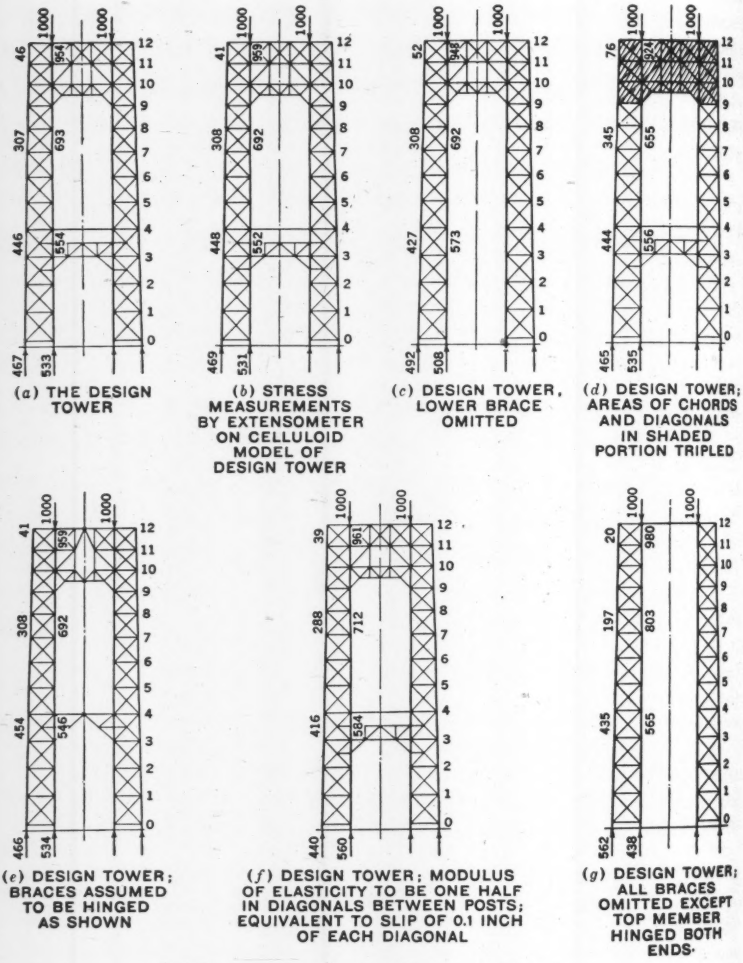

(a) THE DESIGN TOWER
(b) STRESS MEASUREMENTS BY EXTENSOMETER ON CELLULOID MODEL OF DESIGN TOWER
(c) DESIGN TOWER, LOWER BRACE OMITTED
(d) DESIGN TOWER; AREAS OF CHORDS AND DIAGONALS IN SHADED PORTION TRIPLED
(e) DESIGN TOWER; BRACES ASSUMED TO BE HINGED AS SHOWN
(f) DESIGN TOWER; MODULUS OF ELASTICITY TO BE ONE HALF IN DIAGONALS BETWEEN POSTS; EQUIVALENT TO SLIP OF 0.1 INCH OF EACH DIAGONAL
(g) DESIGN TOWER; ALL BRACES OMITTED EXCEPT TOP MEMBER HINGED BOTH ENDS

FIG. 4.—VARIATIONS IN DESIGN OF TOWERS.

It was thought desirable, therefore, to test the sensitivity of the tower frame by varying the assumed elastic conditions of its various members and the character and proportions of the braces that connect the two bents. In

this way, a measure was obtained of the sensitiveness of the stresses in the tower to such variations. These studies furnished interesting information as to the behavior of statically indeterminate structures of this type and showed their inherent stability. A number of these studies are shown on Fig. 4. The numbers in all cases indicate the distribution, in the various members of the frame, of the stress caused by two symmetrically applied load reactions of 1 000 units each. They show clearly how the loads travel down the frame to the pier and how they are fed into the outer posts.

Fig. 4(*a*) represents the frame of the design finally adopted. It shows the stresses produced by the loads at several elevations and the distribution between the inner and outer posts, until at the pedestals the outer posts attain a reaction of 46.7% of the ideal 50 per cent.

Fig. 4(*c*) shows the frame of the tower with the lower brace omitted, so that the tower represents a frame well braced on top and resting on the tower foundation. In this case the variations in the stresses and reactions and those computed for the design tower shown in Fig. 4(*a*), are less than 3% throughout. The distribution of the load on the pier here becomes nearly ideal for vertical forces.

Fig. 4(*d*) shows the tower frame as designed, except that the areas of the top braces are assumed to be tripled; in other words, in Fig. 4(*d*), the top portal is three times as rigid as the one adopted. The stress distribution in this frame shows that the stresses and pier reactions vary from those in the design tower less than 4 per cent. This effect appears in the upper part of the tower and is almost lost in the lower part. It serves to prove the sufficient rigidity of the top braces as built and further proves that increased rigidity would improve the distribution of the stresses only slightly.

Fig. 4(*e*) shows a tower frame with all three braces hinged so that they cannot transmit any bending moments. The variations in the stresses and reactions of this frame have been found to vary from those of the design tower (Fig. 4(*a*)) less than 1 per cent.

Fig. 4(*f*) represents the tower as designed except that the assumption was made here that the modulus of elasticity of all diagonals between the posts is only one-half its value; or (what is the same), that their respective sectional areas are only one-half those in the design tower. This is also equivalent to a slip of 0.1 in. in the riveted connection at one end of each diagonal. The variations in the stresses and reactions resulting from this assumption are not more than 3% from those in the design tower. The studies of the effect of diagonals on the stress distribution are of importance in the design, because here the connections are affected by rivets only and are not made on abutting ends.

Fig. 4(*g*) shows a frame with the simplest and most flexible connection between the two bents. All the braces are replaced by a single strut at the top, hinged at both ends. This is a frame statically indeterminate in the first degree. It represents the lowest extreme variation in the bracing. Even in this case of greatest flexibility the maximum variation of the stresses from those of the design tower does not exceed 19 per cent.

To check the stress distribution roughly, the tower may be considered as consisting of two tower bents for the lower nine panels, with a comparatively rigid truss resting on the top of these bents. The reaction applied on top of this truss would be distributed by it to the posts in the same manner as any truss transmits loads to its piers. An analysis of the stresses from this point of view showed that the truss as actually proportioned could safely transmit at least 40% of the load to the outer posts, whereas the elastic computations on which the design is based, require only about 30% to be transmitted at that point. Pushing this gross assumption further by assuming that all the diagonals in the bents in the lower nine panels have been omitted, without affecting the stability of the tower, the outer posts would thus sustain 40% and the inner posts 60% of the reaction at the top. Actually, the outer posts are designed for a stress varying from 39% at Point 9 to 61% at the pedestal, and the inner posts for a stress from 70% at Point 9 to 61% at the bottom.

To obtain a comprehensive picture of the results of these studies similar to what the biologist would term "the mutation of the species," the resulting stresses were plotted as shown in Fig. 5. In the same diagram is shown the

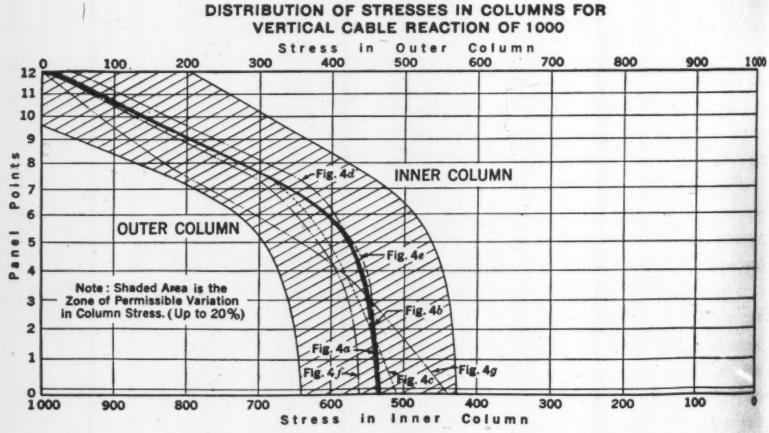


FIG. 5.—CURVES SHOWING RELATIVE DISTRIBUTION OF A VERTICAL CABLE REACTION OF 1 000 BETWEEN INNER AND OUTER COLUMNS OF TOWER (SEE FIG. 4).

hatched zone enclosing a variation of stress of 20 per cent. It may be called the zone of permissible stress variations from computed theoretical stress values. Within this zone are found the extremely improbable and practically impossible variations that have been the objectives of this study.

The studies of the efficiency of the design show definitely that although the tower is statically indeterminate to a high degree, it is definite and steady in its behavior and is little disturbed by variations from the assumptions on which the elastic computations are based.

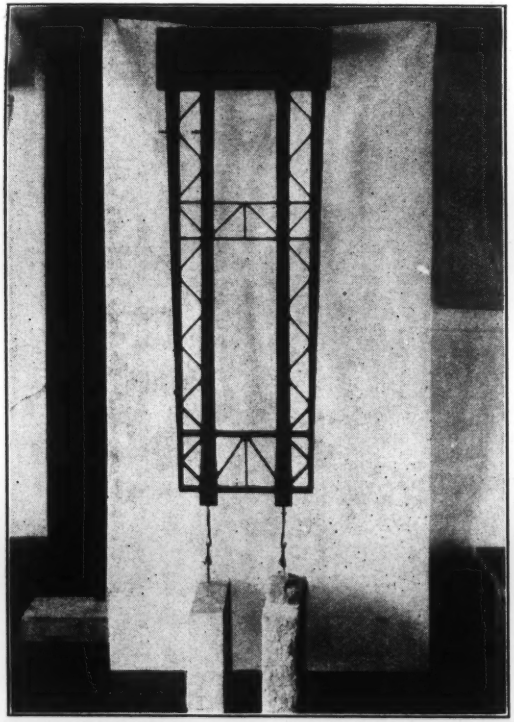


FIG. 6.—CELLULOID MODEL OF TOWER, INVERTED AND SUBJECT TO TENSILE LOADS.

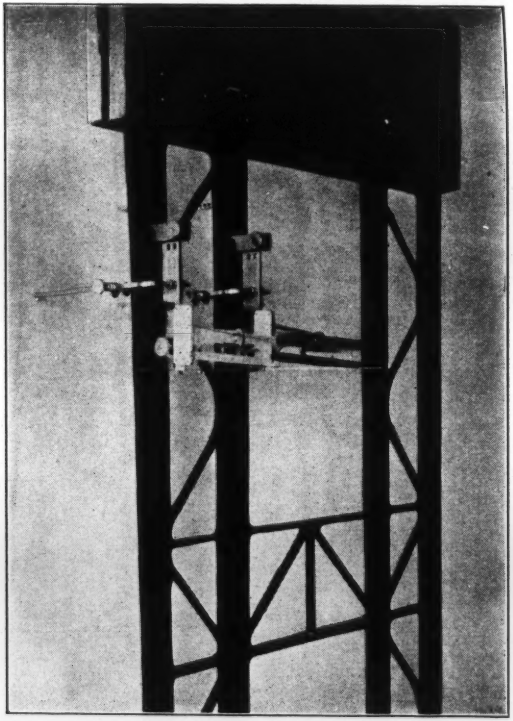


FIG. 7.—TENSOMETERS IN POSITION DURING TEST

Stress-Strain Measurements on Model of Tower

As indicated by the foregoing studies, the design of the steel towers for this bridge presented a novel feature in the planning of towers for large bridges under special conditions. The analytical treatment of the towers involves the handling of many unknowns and requires elaborate arithmetical computations. It was deemed desirable, therefore, to present a physical object and to obtain sufficient verification of the action of the frames so that their elastic behavior under load could be demonstrated. As the main object of the test was to observe the distribution of stress between the inner and outer posts, it was sufficient to consider one of the four frames only. Such a frame could not act as a compression member, of course, and would fail in buckling. It was concluded, therefore, that the object of the test would be attained if the frame were to be reversed in position, with the top down, and the reactions applied to it as loads, thus putting the entire frame in tension. Such a model should be of a uniform homogeneous material, preferably of one piece. It should behave elastically the same as a complete structure, or as nearly as it can be made to do so.

Celluloid was selected for the model because of its uniformity and because of the ease with which it can be cut. The largest sheet of celluloid obtainable was 50 by 20 in. and the scale of the model was made accordingly. The dimensions of the full-sized frame were thus reduced by 140; that is, each foot measured along the center lines of the model corresponded to 140 ft. of the actual structure. The areas were treated in a different manner. Within certain limits the width of the members effects the secondary stresses only, and if the strain measurements are taken along the center lines, their influence on the secondary stresses vanishes. It was considered advisable, therefore, to keep the same thickness for all the members and to vary the width only according to the area. In this way, the model could be made in one piece of constant thickness. For further simplification, the double diagonals between the posts were replaced by a single set, and the two top braces were represented by one of a corresponding moment of inertia. These simplifications do not change the elastic behavior of the structure. To keep the width of the smallest members within practical dimensions, it was found best to represent 1 000 sq. in. of the full-sized structure by 1.6 in. of width in the model, the thickness for all model members being 0.25 in. Therefore, 1 000 sq. in. of steel corresponded to 0.4 sq. in. of the model. The reduction in area thus was 1 : 2 500.

Incidentally, the celluloid model presented a good picture of the distribution of the material in the actual structure because the width of the members was proportional to their cross-sectional areas. To eliminate, entirely, any possibility of longitudinal flexure from compressive forces, the model was inverted, allowed to swing freely, and the load reactions were applied at the proper places as tensile forces, as shown in Fig. 6. It was subjected to two equal loads of 62.5 lb. which produced a greatest unit stress in the frame of 108 lb. per sq. in. This stress is well within the elastic limit of the celluloid.

The instruments used to measure the distribution of stresses are based on Hooke's law; namely, the elongation of the material is assumed to be in direct proportion to its unit stresses. Two tensometers with a magnification of 1 000 were used with very satisfactory results (see Fig. 7). The instruments are easy to handle and give sufficiently accurate results.

Two instruments were placed at the same level, one on the center line of the inner posts and the other on the center line of the outer posts of the same side of the model. Both had a gauge length of 1 decimeter (3.94 in.). The first tensometer readings were taken with no load. The model was then subjected to the two loads, and the second set of readings was taken. The load was removed, and a third set of readings was recorded. These corresponded closely to the first readings. The elongation of the first tests is the average of the foregoing differences; that is, the difference between the first and the second readings and the difference between the second and third readings. To obtain a fair average the test was repeated four times.

The elongations were magnified 1 000 times and the distances observed attain an average of about 1 in., a difference large enough to derive close results. The results of the observations made on the model have shown

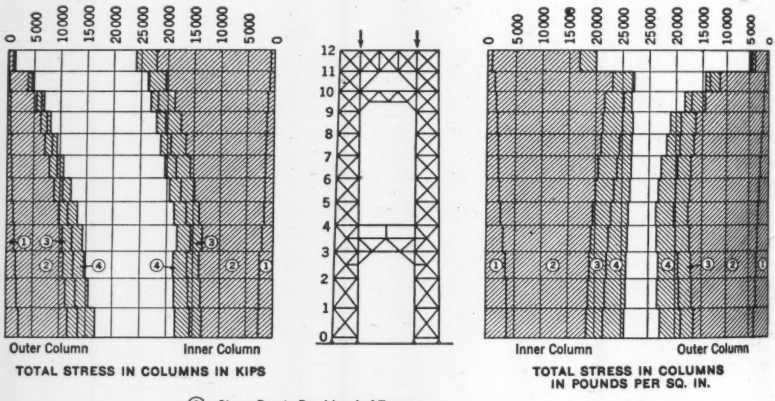

0 5000 10000 15000 20000 25000 20000 15000 10000 5000 0
Outer Column
Inner Column
TOTAL STRESS IN COLUMNS IN KIPS
12 11 10 9 8 7 6 5 4 3 2 1 0
0 5000 10000 15000 20000 25000 25000 20000 15000 10000 5000 0
Inner Column
Outer Column
TOTAL STRESS IN COLUMNS IN POUNDS PER SQ. IN.
① Stress Due to Dead Load of Tower
② Stress Due to Dead Load of Suspended Span
③ Stress Due to Live Load and Temperature Vertical Cable Reaction
④ Stress Due to Live Load and Temperature - Bending

FIG. 8.—STRESS IN TOWER COLUMNS.

stresses in close agreement with those computed. As shown in Fig. 4(*b*), the variation between the observed and computed stresses was found to be less than 1 per cent.

STRESS DISTRIBUTION IN TOWER COLUMNS

To obtain a clear picture of the relative values of the stresses produced in the twelve tower panels by the various loads, the diagrams shown in Fig. 8 were drawn. The left side represents the total stresses in the outer and inner

columns that are produced by the dead load of the towers, the dead load of the suspended span, the live load and the temperature due to the vertical cable reaction, and the live load and the temperature due to bending. The right side represents the unit stresses due to these respective causes. The diagrams in Fig. 8 show at sight the duties that the tower members must perform to sustain the forces that act on the tower. From the diagram of the unit stresses the relative importance can also be seen of the action of these forces.

Strain Measurements

The determination of the stresses by the analytical process, the studies of the elastic behavior and stability of the frames by what may be termed scholastic methods of reasoning, and the observations made on a simple model with the aid of modern apparatus, were all means of predicting the stress-strain behavior of the tower. They are all available tools of science, reason, and experience to foretell the action of a structure designed and erected in accordance with certain premises. They furnished sufficient concording information to heighten confidence and trust in the stability and the safety of the planned towers. It remained for the engineers to verify by observation how the structure would actually behave when built. This was done by a program of extensive strain measurements on various parts of the erected towers.

A series of strain measurements were conducted on the south leg of the New York tower at various stages of erection and when the bridge was completed and ready for its initial traffic. Because the time available for taking measurements at any one stage was limited due to the progress of construction, it was necessary to select as few members for this purpose as would give a clear picture of the path of the load from the saddles to the base of the tower. The members selected are shown in Fig. 9 in which, of course, the bents are separated and the connecting braces omitted. The location of a member in the particular bent and panel was designated by the letters given in Fig. 10(*a*). For horizontals the second number designates the panel point instead of the panel. The New York tower was chosen because construction on that tower was begun later than on the New Jersey tower, and this gave more time to complete the preliminary work necessary for the strain measurements.

Two series of gauge lines were used—primary and secondary. The primary lines were for the purpose of determining the direct or average stresses on the sections, and the secondary lines for determining the secondary bending stresses, or the distribution of stress across the observed sections. The primary series consisted of 20-in. gauge lines in all the members selected. The lines were located as nearly as possible to the middle of each member, care being taken to keep them away from reinforcing plates, splices, or other connections that might introduce local stresses.

Fig. 10(*b*) shows the location of the individual gauge lines in the section of the tower columns. Lines 1 to 8 at the mid-height of the column are primary gauge lines; Lines 1 to 8 at the top and bottom of the column are

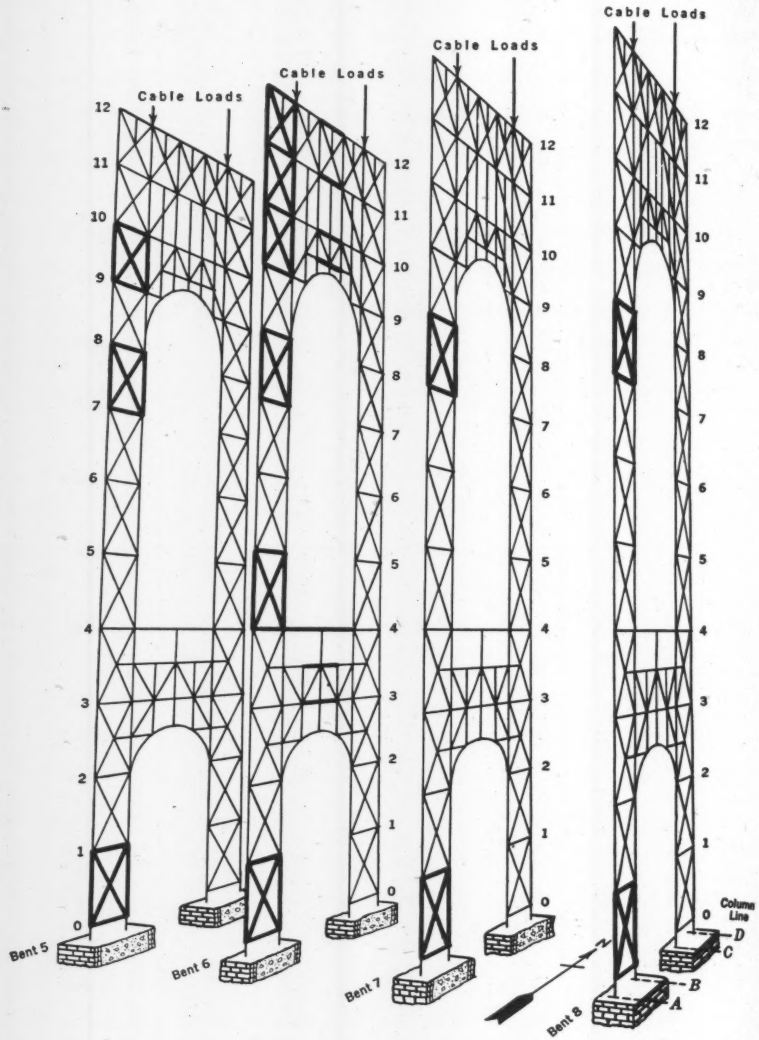


FIG. 9.—MEMBERS OF NEW YORK TOWER SELECTED FOR STRESS IN MEASUREMENTS.

secondary gauge lines; and Lines 9 to 20, inclusive, are secondary gauge lines. This same number and arrangement of lines was used in all the columns measured, the only exception being in the *B*-columns (see Fig. 9) in Panels 11 and 12. These columns have three and five cells, respectively, instead of one cell, as in the others. In these two cases each cell was treated as an individual column, and the same number and arrangement of gauge lines was used in each cell as in the other columns.

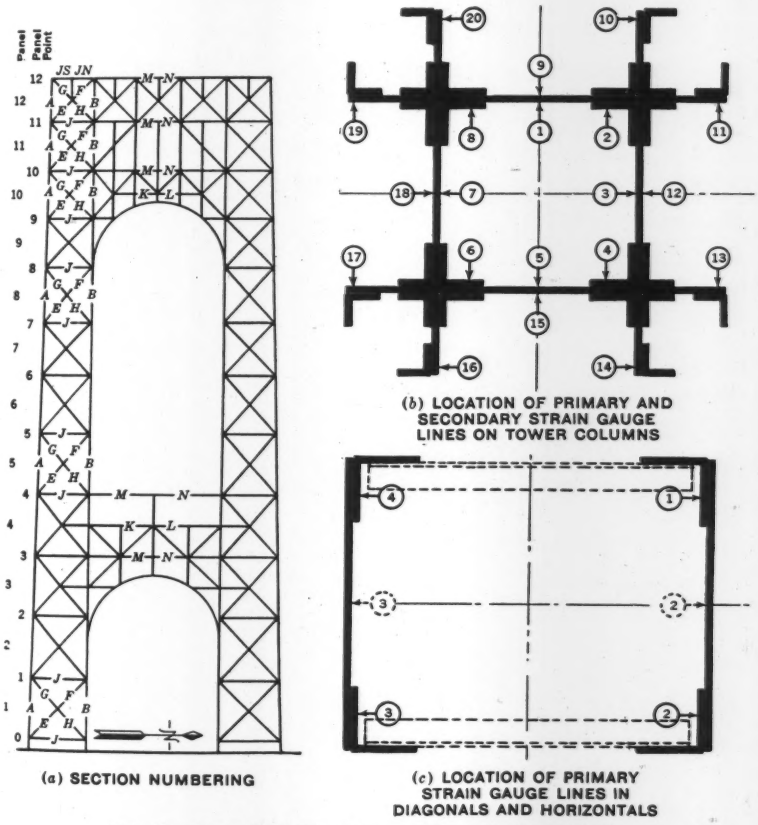

Panel Point
Panel
JS JN
M N
K L
(a) SECTION NUMBERING
(b) LOCATION OF PRIMARY AND SECONDARY STRAIN GAUGE LINES ON TOWER COLUMNS
(c) LOCATION OF PRIMARY STRAIN GAUGE LINES IN DIAGONALS AND HORIZONTALS

FIG. 10.—GAUGE LINE SYMBOLS FOR STRESS MEASUREMENTS.

Fig. 10(*c*) shows the location of the individual gauge lines in the sections of the diagonals and horizontals. The only variation in this arrangement occurred in the deep horizontals in Panel 12 and in the top portal where an additional gauge line was placed in the middle of the web on each side. The gauge lines shown dotted were additional ones assigned to deep horizontals in Panel 12 and in the top portal.

The secondary gauge lines were placed on the *B*-columns in Panel 1. They were 10-in. lines located at the top and bottom of each column, both inside and outside (see Fig. 10(*b*)). To determine Poisson's ratio an additional set of four 10-in. gauge lines was placed in Column 51*B* at right angles to the primary set at that point to measure the strain at right angles to the axis of the column. In addition, a set of 20-in. gauge lines was placed on the outside of each *B*-column in Panel 1 opposite the 20-in. primary set on the inside. All these lines were located so as to minimize the effect of local stresses from splices or connections.

TABLE 1.—Comparison of Computed Stresses with Those Measured by Strain Gauges

(Units are kips, or thousands of pounds, per square inch)

M or C*	Columns							Diagonals							Horizontals						
	Member †	Stages ‡						Member †	Stages ‡						Member †	Stages ‡					
		1	2	3	4	5			1	2	3	4	5			1	2	3	4	5	
M	51A	−5.1	−10.4	−12.6	−18.0	−14.4		81H	−1.4	−1.0	−3.0	−1.2	−1.6		50J	+2.0	.....	+2.7	+2.7	+1.0	
C		−4.5	−10.3	−12.4	−17.1	−13.4			−0.4	−0.8	−2.2	−1.8	−3.4			+0.2	.....	+0.3	+1.2	0	
M	61A	−3.9	−9.0	−11.4	−15.0	−14.6		65E	−2.1	.....	−10.5	.....	−12.8		60J	+0.9	.....	+1.5	+1.0	0	
C		−4.0	−8.6	−11.5	−14.5	−13.4			.....	.....	.....	.....	.....			+0.1	.....	+0.1	+0.4	0	
M	71A	−3.4	−7.2	−11.0	−10.2	−15.0		65F	−2.4	.....	−11.1	.....	−13.2		70J	+2.0	.....	+3.0	+2.7	+0.6	
C		−3.4	−7.0	−10.7	−11.9	−13.4			−2.5	.....	−10.5	.....	−13.0			−0.1	.....	−0.1	+0.4	0	
M	81A	−1.4	−4.5	−9.6	−12.3	−13.6		65G	+0.5	.....	+2.6	.....	+3.8		80J	+3.4	.....	+3.0	+3.0	+2.0	
C		−3.0	−5.4	−9.3	− 9.4	−13.5			.....	.....	.....	.....	.....			−0.2	.....	−0.3	+1.2	0	
M	51B	−6.2	−11.7	−14.1	−19.8	−14.7		65H	+0.3	.....	+2.6	.....	+2.8		51J	+2.8	.....	+7.5	+9.9	+8.4	
C		−5.0	−12.0	−14.4	−19.6	−15.5			+0.3	.....	+2.8	.....	+3.7			+2.6	.....	+7.3	+11.1	+8.0	
M	61B	−3.8	−8.7	−12.4	−15.8	−14.8		58E	−1.6	−6.3	−9.2	.....	−12.2		61J	+1.2	.....	+7.2	+9.0	+8.2	
C		−4.5	−10.0	−13.4	−16.7	−15.5			.....	.....	.....	.....	.....			+2.3	.....	+6.9	+8.7	+8.0	
M	71B	−2.6	−6.4	−11.2	−12.2	−13.8		58F	−1.8	−6.3	−9.2	.....	−12.4		71J	+2.1	.....	+7.0	+7.5	+8.4	
C		−4.1	−8.7	−12.3	−13.8	−15.5			−2.1	−6.2	−8.9	.....	−11.2			+2.0	.....	+6.4	+7.2	+8.0	
M	81B	−2.2	−5.7	−11.2	−10.5	−15.0		58G	0	+0.6	+1.5	.....	+2.7		81J	+1.6	.....	+6.0	+6.4	+9.0	
C		−3.6	−6.4	−11.3	−11.0	−15.6			.....	.....	.....	.....	.....			+2.6	.....	+6.0	+5.7	+8.1	
M	65A	−3.2	.....	−11.6	.....	−14.4		58H	+0.3	+1.0	+2.0	.....	+2.7		64J	+1.6	.....	+6.0	.....	+7.5	
C		−3.2	.....	−11.4	.....	−13.8			+0.5	+1.9	+3.0	.....	+3.6			+1.3	.....	+5.5	.....	+6.8	
M	65B	−3.6	.....	−12.4	.....	−14.8		68E	−1.6	−5.8	−8.8	.....	−11.7		65J	+1.8	.....	+7.2	.....	+8.6	
C		−3.2	.....	−12.6	.....	−15.4			.....	.....	.....	.....	.....			+1.5	.....	+5.8	.....	+7.1	
M	58A	−1.6	−6.3	−9.2	.....	−11.7		68F	−1.6	−6.2	−9.4	.....	−12.6		57J	+1.6	+5.1	+6.6	.....	+8.1	
C		−2.8	−7.6	−10.6	.....	−12.6			−2.0	−6.1	−9.0	.....	−11.3			+1.4	+4.0	+5.4	.....	+6.8	
M	68A	−1.5	−6.3	−9.6	.....	−12.3		68G	+1.2	+2.7	+3.3	.....	+4.0		67J	+2.1	+5.0	+6.3	.....	+8.1	
C		−2.5	−7.2	−10.4	.....	−12.7			.....	.....	.....	.....	.....			+1.3	+3.8	+5.5	.....	+6.9	
M	78A	−2.1	−6.6	−10.5	.....	−13.4		68H	+1.6	+3.0	+3.0	.....	+3.6		77J	+1.8	+4.8	+6.6	.....	+8.4	
C		−2.2	−6.8	−10.2	.....	−12.8			+0.6	+1.9	+2.9	.....	+3.6			+1.1	+3.7	+5.6	.....	+7.0	
M	88A	−0.3	−5.0	−8.8	.....	−12.8		78E	−1.0	−5.4	−8.2	.....	−11.0		87J	+1.2	+4.2	+6.0	.....	+8.7	
C		−1.9	−6.3	−10.0	.....	−12.9			.....	.....	.....	.....	.....			+1.0	+3.5	+5.7	.....	+7.0	
C	58B	−2.7	−8.6	−12.6	.....	−15.6		78F	−0.4	−5.6	−8.2	.....	−11.1		58J	−0.2	+2.2	+2.7	.....	+4.0	
C		−2.8	−8.8	−12.2	.....	−15.7			−1.9	−6.1	−9.1	.....	−11.3			+1.4	+4.0	+5.5	.....	+7.2	
M	68B	−2.8	−8.8	−12.4	.....	−16.5		78G	−0.2	+2.7	+3.3	.....	+3.2		68J	+1.2	+3.3	+5.0	.....	+6.0	
C		−2.6	−8.6	−12.5	.....	−15.8			.....	.....	.....	.....	.....			+1.3	+4.0	+5.7	.....	+7.3	
M	78B	−3.0	−8.7	−13.0	.....	−17.1		78H	+1.2	+2.7	+3.6	.....	+3.2		78J	+1.8	+4.0	+5.4	.....	+6.9	
C		−2.4	−8.3	−12.8	.....	−15.9			+0.7	+2.0	+2.8	.....	+3.6			+1.2	+4.0	+6.0	.....	+7.3	
M	88B	−2.2	−8.4	−13.5	.....	−17.2		88E	−1.2	−5.8	−8.8	.....	−11.7		88J	+1.6	+3.0	+4.0	.....	+6.0	
C		−2.2	−8.1	−13.1	.....	−16.0			.....	.....	.....	.....	.....			+1.0	+3.9	+6.2	.....	+7.4	
M	510A	−2.0	.....	−8.2	.....	−10.8		88F	−1.2	−5.4	−8.7	.....	−11.1		59J	+0.3	.....	+3.9	.....	+5.7	
C		−1.9	.....	−8.5	.....	−10.7			−1.8	−6.0	−9.2	.....	−11.3			+0.5	.....	+3.8	.....	+5.3	
M	510A	−2.8	.....	−12.0	.....	−15.3		88G	+1.5	+2.1	+2.7	.....	+2.8		69J	+0.9	.....	+5.2	.....	+7.3	
C		−2.2	.....	−11.6	.....	−15.8			.....	.....	.....	.....	.....			+0.5	.....	+4.0	.....	+5.3	
M	610A	−1.6	.....	−8.8	.....	−11.2		88H	+1.2	+2.1	+2.6	.....	+2.6		510J	+0.2	.....	+5.6	.....	+7.6	
C		−1.8	.....	−8.6	.....	−10.8			+0.8	+2.0	+2.7	.....	+3.5			+1.4	.....	+6.7	.....	+9.3	
M	610B	−3.6	.....	−12.8	.....	−16.5		510E	−1.6	.....	−6.0	.....	−8.7		610J	−1.2	.....	+4.8	.....	+7.2	
C		−2.1	.....	−12.3	.....	−15.9			.....	.....	.....	.....	.....			+1.4	.....	+7.2	.....	+9.1	
M	611A	−1.8	.....	−6.4	.....	−10.4		510F	−1.5	.....	−6.6	.....	−8.4		611J	+0.3	.....	+3.9	.....	+5.6	
C		−1.3	.....	−6.6	.....	−8.4			−1.1	.....	−6.9	.....	−9.2			+0.4	.....	+3.8	.....	+5.1	
M	611B	−1.5	.....	−11.5	.....	−15.4		510G	+0.4	.....	+3.0	.....	+4.0		612JN	−0.2	.....	−2.7	.....	−3.8	
C		−1.6	.....	−11.1	.....	−14.6			.....	.....	.....	.....	.....			.....	.....	.....	.....	.....	
M	612A	−0.3	.....	−2.4	.....	−3.3		510H	+0.3	.....	+2.4	.....	+3.9		612JS	−0.3	.....	−2.2	.....	−2.7	
C		−0.5	.....	−2.4	.....	−3.0			0	.....	+3.2	.....	+3.7			−0.2	.....	−2.4	.....	−3.0	
M	612B	−1.4	.....	−9.1	.....	−12.0		610E	−1.6	.....	−6.9	.....	−9.3								
C		−1.1	.....	−9.0	.....	−12.1			.....	.....	.....	.....	.....								

* M denotes measured stresses; C denotes computed stresses.
† Numerals and letter indicate the position of the member in the tower; thus, see, Figs. 9 and 10 (*a*), 50*J*, indicates that the member is the horizontal, *J*, at Panel Point 0, in Bent 5.
‡ See Fig. 11.

TABLE 1.—(*Continued*)

M or C*	Diagonals: Member †	Diagonals: Stages ‡ 1	2	3	4	5	Diagonals: Member †	Diagonals: Stages 1	2	3	4	5	Horizontals in Braces: Member †	Horizontals in Braces: Stages ‡ 1	2	3	4	5
M	51E	−2.7	−6.8	−7.2	−10.5	−9.8	610F	−1.8	.....	−6.9	.....	−9.3	63K	.....	.....	+1.6	.....	+1.8
C								−1.0	.....	−7.2	.....	−9.2		.....	.....	.....	.....	.....
M	51F	−2.6	−6.4	−6.3	−10.5	−9.9	610G	+0.2	.....	+3.6	.....	+4.8	63L	.....	.....	+1.6	.....	+2.4
C		−3.0	−6.4	−7.9	−10.0	+8.8			.....		.....			.....	.....	+1.0	.....	+1.3
M	51G	−0.3	−0.9	−2.4	−2.2	−0.3	610H	−0.2	.....	+2.7	.....	+4.2	63M	.....	.....	+0.9	.....	+3.0
C								0	.....	+2.9	.....	+3.6		.....	.....	.....	.....	.....
M	51H	−0.9	−1.5	−2.8	−3.0	−1.2	611E	−1.4	.....	−10.4	.....	−14.8	63N	.....	.....	+1.6	.....	+1.8
C		−0.8	−2.8	−3.2	−4.8	−3.3			.....		.....			.....	.....	+1.4	.....	+1.2
M	61E	−2.4	−5.7	−6.2	−9.8	−9.6	611F	−1.4	.....	−10.6	.....	−14.7	64K	.....	.....	+1.8	.....	+2.2
C								−1.6	.....	−10.3	.....	−13.2		.....	.....	.....	.....	.....
M	61F	−2.0	−5.1	−6.2	−9.0	−9.3	611G	+0.6	.....	+6.2	.....	+8.1	64L	.....	.....	+2.4	.....	+3.0
C		−2.9	−5.7	−7.6	−9.3	−8.9			.....		.....			.....	.....	+1.1	.....	+1.3
M	61G	−0.2	−1.4	−2.8	−2.6	−1.4	511H	+0.6	.....	+5.8	.....	+8.4	610K	0	.....	+2.4	.....	+3.8
C								+0.9	.....	+7.0	.....	+8.6		.....	.....	.....	.....	.....
M	61H	−1.2	−1.2	−2.6	−2.6	−1.4	612E	−1.8	.....	−10.8	.....	−15.0	610L	+0.6	.....	+2.2	.....	+4.2
C		−0.7	−2.1	−2.9	−3.8	−3.4			.....		.....			+0.6	.....	+2.0	.....	+2.4
M	71E	−2.1	−5.4	−6.0	−8.6	−10.4	612F	−0.4	.....	−10.2	.....	−13.6	610M	−0.3	.....	+1.4	.....	+2.4
C								−1.4	.....	−11.0	.....	−14.2		.....	.....	.....	.....	.....
M	71F	−1.6	−5.0	−6.2	−8.1	−10.0	612G	+0.9	.....	+8.0	.....	+10.6	610N	−0.3	.....	+4.0	.....	+2.4
C		−2.7	−5.0	−7.3	−8.2	−8.9			.....		.....			+0.1	.....	−2.4	.....	+5.1
M	71G	−0.8	−1.5	−2.8	−1.8	−2.0	612H	+1.2	.....	+7.0	.....	+9.9	611M	−0.9	.....	−2.4	.....	−2.7
C								+0.8	.....	+7.5	.....	+9.3		.....	.....	.....	.....	.....
M	71H	−1.0	−1.5	−2.2	−2.4	−2.1	.....	.....	.....	.....	.....	.....	611N	−0.6	.....	−2.7	.....	−2.4
C		−0.5	−1.4	−2.6	−2.8	−3.4	.....	.....	.....	.....	.....	.....		−0.3	.....	−2.1	.....	−2.8
M	81E	0	−2.7	−5.1	−6.3	−8.8	.....	.....	.....	.....	.....	.....	612M	−1.4	.....	−4.2	.....	−5.0
C							.....	.....	.....	.....	.....	.....		.....	.....	.....	.....	.....
M	81F	−0.3	−2.8	−4.6	−6.2	−9.0	.....	.....	.....	.....	.....	.....	612N	−1.2	.....	−3.6	.....	−4.5
C		−2.5	−4.3	−7.0	−7.2	−8.9	.....	.....	.....	.....	.....	.....		−0.8	.....	−4.5	.....	−5.6
M	81G	−0.3	−0.5	−2.0	−0.9	−1.4	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
C		.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

* M denotes measured stresses; C denotes computed stresses.
† Numerals and letter indicate the position of the member in the tower; thus, see, Figs. 9 and 10 (*a*), 50 *J*, indicates that the member is the horizontal, *J*, at Panel Point 0, in Bent 5.
‡ See Fig. 11.

Gauge Holes

A gauge line consists of two holes spaced the proper distance apart. To keep the distance within proper limits magnetic templets were used. A 6-volt storage battery supplied the current for the templet. The gauge holes were drilled through the templets, by means of a No. 55 combination drill and a 60° countersink of high-speed steel. The drill was run by a ¼-h. p. electric motor and flexible shaft. After a hole was drilled, and before removing the templet, it was finished by inserting a hardened steel pin with a point having the same angle (55°) as the legs of the strain gauge. Striking this pin lightly with a hammer smoothed out any irregularities or fins and made a good seat for the legs of the strain gauge. The holes were kept filled with vaseline and covered with adhesive tape. The gauge holes were drilled while the members were in the railroad yard or as soon after they were erected as possible.

Measuring Equipment

The strain gauges operated by hand were of an improved form designed by R. S. Johnston, M. Am. Soc. C. E. Combined with a 10 to 1 lever the dial read directly to 0.0001 in. and was easily estimated to one-tenth of a

division. The range of the gauge was 0.04 in. The gauges were calibrated throughout their entire range at various intervals by means of a micrometer microscope, which in turn was calibrated against a standard scale. Three strain gauges were used, one having a 10-in., and two having a 20-in. gauge length. Temperatures were read by mercury as well as by resistance thermometers.

Method of Taking Readings

At sections where readings were taken fixed platforms were built for that purpose. Movable ladder jacks and platforms were used inside the columns. All readings were taken at night because exposure to the sun caused uneven temperatures in the members during the day. In all cases strain-gauge readings were never taken until at least three hours after sunset. The stresses deduced from the strain measurements were based on a modulus of elasticity of 30 000 000. The average modulus of elasticity for twenty-nine test specimens from material representing the columns was 29 800 000. The minimum value was 28 800 000 and the maximum, 31 000 000. The study of the reliability of the measurements obtained led to the conclusion that as a whole the errors in the measured results were within the following ranges: For diagonals and horizontal, $\pm$ 200 lb. per sq. in., with a maximum of 800 lb. per sq. in.; and, for columns, $\pm$ 500 lb. per sq. in., with a maximum of 1 300 lb. per sq. in.

Stages of Loading

Strain measurements were made for five successive stages during the construction of the bridge, which meant five different conditions of loading: (1) With the completed tower and catwalk, or temporary platform, for stringing the cables; (2) Stage 1 with the four cables completed; (3) Stage 2, including the floor steel of the suspended structure; (4), Stage 3, including the completed floor-slab in the main span and three-fourths of that of the side span; and (5) the completed bridge with the single deck in addition to the catwalk. The tower saddles were then in their final position for this stage. The members measured at the various stages are shown in Fig. 11.

The measured and computed stresses for all the sections, for the five stages, are given in Table 1. The measured stress reported is the average for all the primary gauge lines in that section.

Columns

The agreement between measured and computed stress in the columns is very close as shown by Figs. 12 and 13. The distribution of the cable reaction between the inner and outer columns of Bent 6 is shown graphically in Fig. 12. The distribution of the load between the eight columns of the tower leg at Panels 1 and 8 is shown in Fig. 13. The distribution of the total

load between the inner and outer columns is given in Table 2 for measured and computed stresses. Strain measurements were taken on one of the columns for the purpose of determining Poisson's ratio. These measurements con-

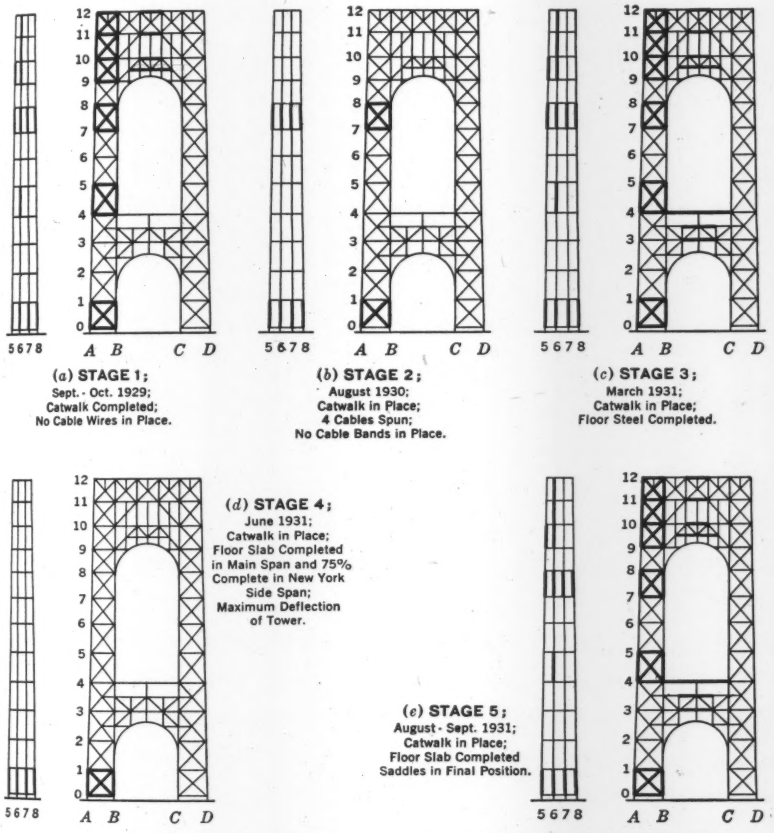


FIG. 11.—DIAGRAMMATIC DESCRIPTION OF STAGES FOR STRAIN-GAUGE MEASUREMENTS.

sisted of one gauge line on each of the four inside plates in Column 51*A*. The results of these measurements are given in Table 3.

DIAGONALS

The difference between the average measured stress and average computed stress for all the diagonals was 370 lb. per sq. in., with a maximum difference of 2 300 lb. per sq. in. The large differences in measured and computed stress usually occurred when the stresses in the diagonals were small.

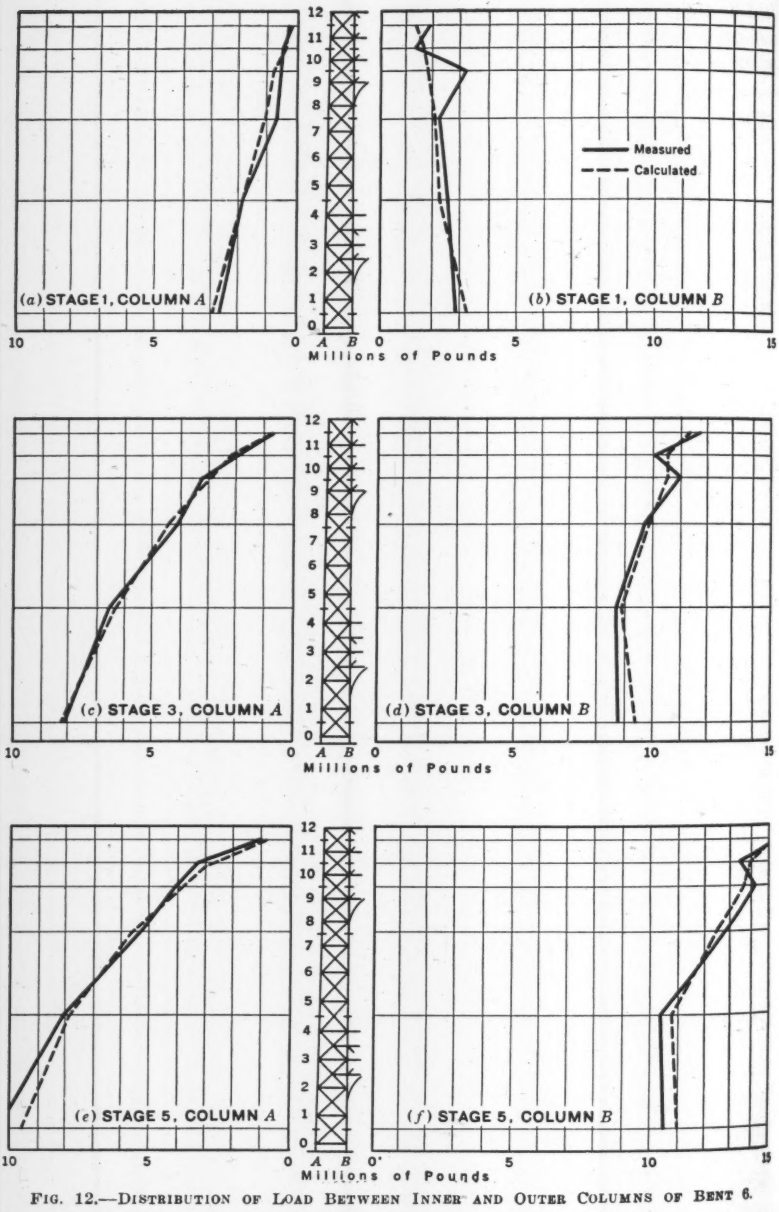


FIG. 12.—DISTRIBUTION OF LOAD BETWEEN INNER AND OUTER COLUMNS OF BENT 6.

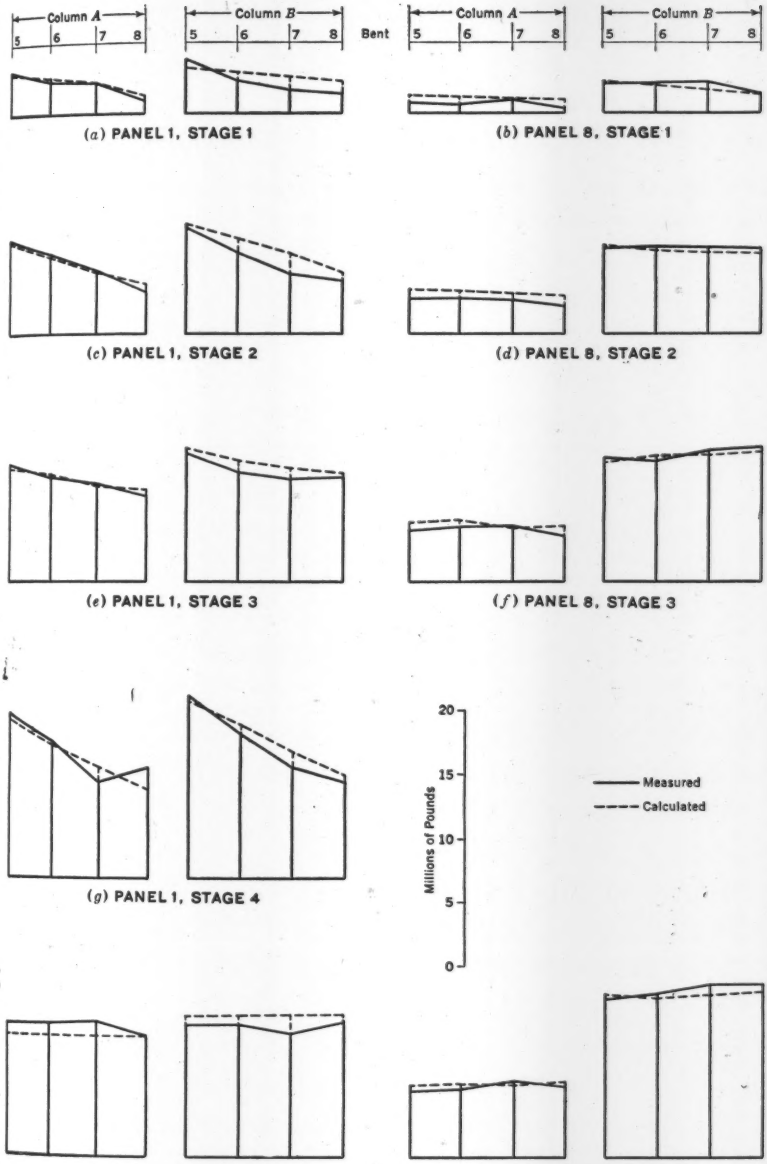


FIG. 13.—DISTRIBUTION OF LOAD BETWEEN EIGHT COLUMNS OF THE TOWER LEG AT PANELS 1 AND 8.

TABLE 2.—Distribution of Load Between Inner and Outer Rows of Columns

Stage	Total Load on Four Outer (*A*) Columns, in Kips		Total Load on Four Inner (*B*) Columns, in Kips		Total Load on Outer and Inner Columns, in Kips	
	Measured	Calculated	Measured	Calculated	Measured	Calculated
			Panel 1			
1	9 884	10 694	10 600	12 316	20 484	23 010
2	22 321	22 463	23 270	26 562	45 591	49 025
3	32 010	31 867	35 016	36 802	67 026	68 669
4	39 833	37 967	41 743	43 748	81 576	81 715
5	41 340	38 540	41 741	44 464	83 081	83 004
			Panel 8			
1	2 426	4 146	8 410	7 859	10 836	12 005
2	10 677	12 302	27 120	26 568	37 797	38 870
3	16 806	18 174	40 479	39 772	57 285	57 946
4	.....	.....	.....	.....	.....	.....
5	22 144	22 496	52 192	49 832	74 336	72 328

Horizontals

The horizontal member at Panel Point 12 had two sets of gauge lines. Both the measured and computed stresses for this member checked closely. The difference between the average measured stress and the average computed stress for all horizontals was 300 lb. per sq. in., with a maximum of 3 600 lb. per sq. in.

TABLE 3.—Measurements to Determine Poisson's Ratio

Item	Primary stress in column	Stress normal to primary stress	Poisson's ratio
1.....	— 6 200	+2 100	0.339
2.....	— 9 200	+3 000	0.326
3.....	—14 100	+4 200	0.298
4.....	—14 700	+4 500	0.303
Average.....	.....	.....	0.316

Results of Stress-Strain Observations

It has been found that in large members built of plates and angles, such as the columns of this bridge, the intensity of stress is not the same for all parts in any section and may vary along the length of any one part without the application of an intermediate external load. This variation may be as much as 4 500 lb. per sq. in.

The average of the stress measured in part of the shapes making up the section does not necessarily indicate the true average stress in the member. However, if one or more gauge lines were placed on every part making up the section a very accurate measurement of the stress at the given section could be made. What is most important, it has been found that the distribution of load between the inner and outer columns at the base is practically equal. It varies from the computed value about 3½% for the completed structure. The agreement between the measured and computed stresses taken as

a whole is very good. The arithmetical average of all the differences was 740 lb. per sq. in. and the algebraic average was 60 lb. per sq. in.

The practically close agreement between the stresses as computed by analysis and those found from extensive and carefully conducted strain measurements on the actual structure as built under five progressive stages of loading is a full justification of the procedure followed in the development of this design. It is one more proof of the correctness of engineering science and reasoning, of the uniformity of the steel used, and of the excellency of its fabrication into members and its erection into the structure.

Indeterminacy.—A few words on indeterminate structures seem to be proper at this point. The term, "statically indeterminate," plainly means that the equations of equilibrium, which are not dependent on the elastic behavior of the structure, are not sufficient to determine the stresses in them. It is easier to analyze a structure that conforms to the purely mathematical conceptions of concentric action, homogeneous material, frictionless hinges, discontinuous members, etc., but such structures do not exist in Nature. Physically, one can only approach them and make allowances for the deficiencies of one's assumptions. Structures that are internally statically indeterminate will be found to be more stable. Correctly analyzed they present a true picture of the acting stresses.

The writer would suggest replacing the term, "statically indeterminate," by the term, "super-static," as more fitting for structures with more members than are merely necessary for equilibrium, so that when a structure requires sixteen elastic equations to be solved to compute its stresses, it may not be considered on first judgment to be sixteen times more unsteady and dangerous. The more that is known of structures the more it is realized that they demand careful and searching study to be designed effectively, and that the ease of computation is no measure of their efficiency.

Stresses and Material

The combination of the loads to be carried by the tower, resulting in greatest stresses is given in Table 4 for both the initial and the final stages. The location of individual members is shown in the framing plan of the tower, Fig. 14, and in Fig. 15. To sustain these large forces two grades of structural steel, which have been used successfully in bridges, were utilized. All main members of the columns were made of silicon steel, except the top members of the outer columns. The bracing (with the exception of a number of diagonals in transverse elevation), all secondary parts, and parts in which stiffness rather than strength was desirable, were made of carbon steel.

In the proportioning of the various members of the towers two distinct loading conditions were considered: One, the initial stage, with the upper deck completed, and the other, the final stage, with the bridge in its final condition and the lower deck added. The initial stage carried 31 400 lb. per ft. of bridge dead load and 4 200 lb. of live load, and the final stage, 39 000 lb. of dead load and 8 000 lb. of live load, resulting in highest cable reactions of 86 000 tons and 112 000 tons, respectively.

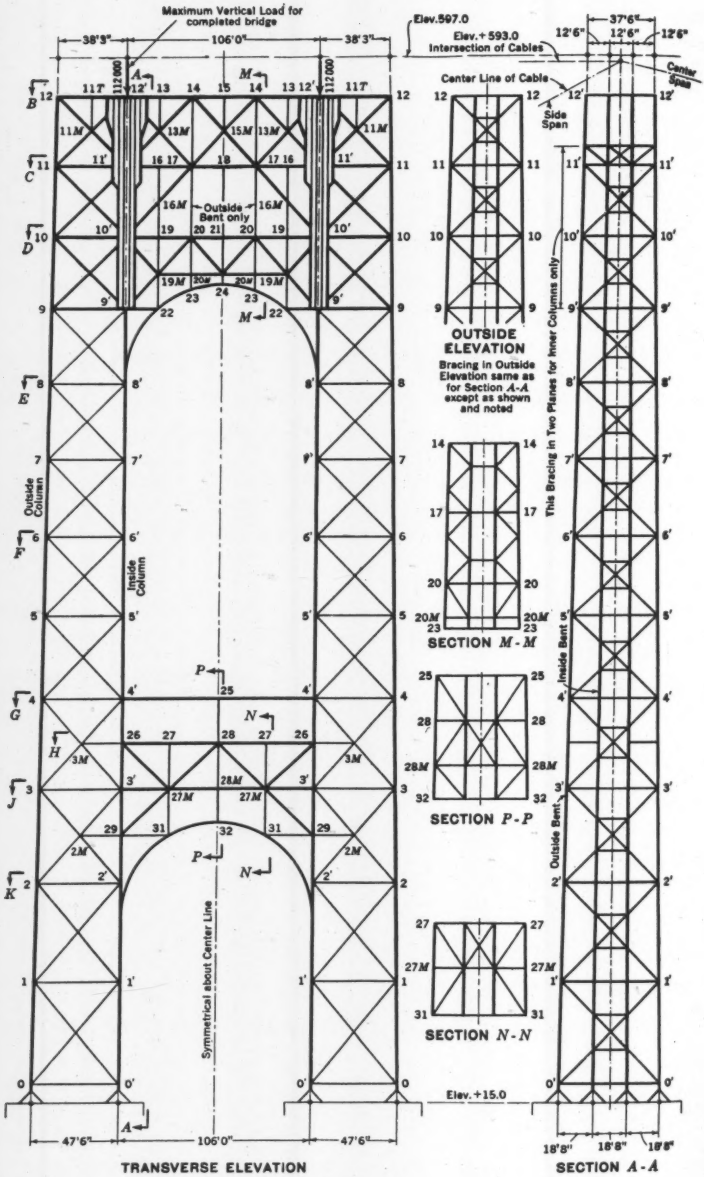


FIG. 14.—TRANSVERSE AND END ELEVATIONS OF TOWER (SEE, ALSO, FIG. 15).

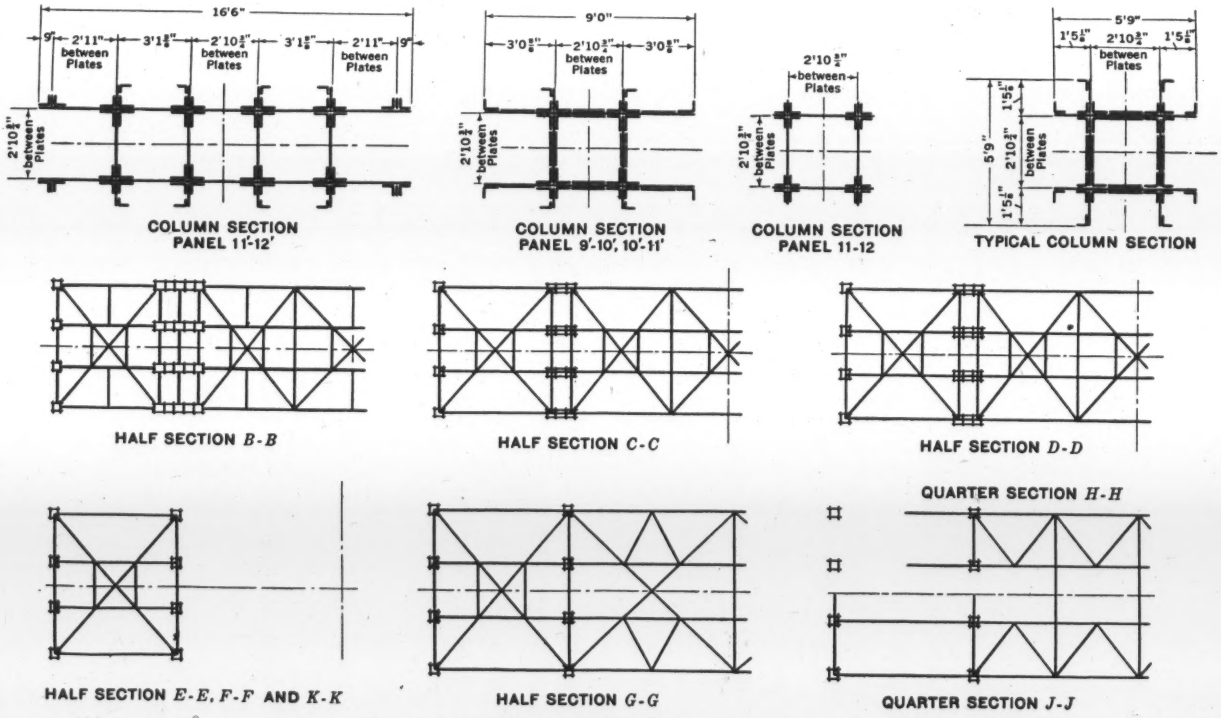


FIG. 15.—DETAILS AND SECTIONS OF TOWER (SEE FIG. 14).

TABLE 4.—Maximum Stresses in the Steel Tower for Initial and Final Stages

(+ denotes tension; − denotes compression; 1 kip = 1 000 lb.)

Member (see Figs. 14 and 15)	Area, in square inches	Initial Stage		Final Stage	
		Stress, in kips	Stress, in kips per square inch	Stress, in kips	Stress, in kips per square inch
Inner Column; Silicon Steel					
0′- 1′	716.0	−15 720	−22.0	−18 450	−25.8
1′- 2′	716.0	−15 680	−21.9	−18 590	−26.0
2′- 3′	716.0	−15 630	−21.8	−18 670	−26.1
3′- 4′	689.8	−14 990	−21.7	−18 020	−26.1
4′- 5′	707.8	−15 380	−21.7	−18 620	−26.3
5′- 6′	733.5	−15 960	−21.8	−19 480	−26.6
6′- 7′	759.3	−16 570	−21.8	−20 320	−26.7
7′- 8′	786.0	−17 150	−21.8	−21 120	−26.9
8′- 9′	812.3	−17 690	−21.8	−21 910	−27.0
9′-10′	860.2	−18 550	−21.6	−23 090	−26.8
10′-11′	877.6	−18 760	−21.4	−23 560	−26.9
11′-12′	1 296.3	−20 380	−15.7	−25 980	−20.0
Outer Column; Silicon Steel					
0- 1	717.7	−14 390	−20.1	−16 860	−23.5
1- 2	674.0	−13 500	−20.0	−15 820	−23.5
2- 3	652.2	−12 590	−19.3	−14 920	−22.9
3- 4	621.5	−12 120	−19.5	−14 460	−23.3
4- 5	575.2	−11 140	−19.4	−13 450	−23.4
5- 6	528.9	−10 060	−19.0	−12 150	−23.0
6- 7	476.4	− 8 940	−18.8	−10 850	−22.8
7- 8	441.4	− 7 830	−17.8	− 9 540	−21.6
8- 9	391.1	− 6 720	−17.2	− 8 240	−21.1
9-10	373.7	− 5 530	−14.8	− 6 840	−18.3
10-11	326.1	− 3 760	−11.5	− 4 660	−14.3
Outer Column; Carbon Steel					
11-12	276.8	− 1 050	− 3.8	− 1 280	− 4.6
Transverse Diagonals; Carbon Steel					
0′ - 1	53.4	− 488	− 9.1	− 628	−11.8
1′ - 2	53.4	− 391	− 7.3	− 517	− 9.7
2′ - 2M	53.4	− 386	− 7.2	− 518	− 9.7
2M- 3	53.4	− 466	− 8.7	− 549	−10.3
3′ - 3M	53.4	− 259	− 4.9	− 275	− 5.2
3M- 4	53.4	− 363	− 6.8	− 488	− 9.1
4′ - 5	53.4	+ 425	+ 8.0	+ 523	+ 9.8
5′ - 6	53.4	+ 438	+ 8.2	+ 540	+10.1
6′ - 7	53.4	+ 439	+ 8.2	+ 543	+10.2
7′ - 8	65.9	+ 430	+ 6.5	+ 532	+ 8.1
8′ - 9	65.9	+ 402	+ 6.1	+ 502	+ 7.6
9′ -10	87.7	+ 454	+ 5.2	+ 574	+ 6.5
10′ -11	102.8	+ 1 189	+11.5	+ 1 505	+14.5
11′ -12	102 8	+ 1 271	+12.4	+ 1 633	+15.8
0 - 1′	53.4	− 815	−15.3	− 1 007	−18.9
1 - 2′	53.4	− 724	−13.6	− 902	−16.9
2 - 2M	53.4	− 730	−13.7	− 914	−17.1
2M- 3′	53.4	− 302	− 5.7	− 377	− 7.1
3 - 3M	68.9	− 655	− 9.5	− 804	−11.7
3M- 4′	68.9	− 904	−13.1	− 1 127	−16.4
Transverse Diagonals; Silicon Steel					
4 - 5′	53.4	− 1 000	−18.7	− 1 209	−22.6
5 - 6′	53.4	− 990	−18.5	− 1 201	−22.5
6 - 7′	53.4	− 982	−18.4	− 1 195	−22.4
7 - 8′	65.9	− 982	−14.9	− 1 198	−18.2
8 - 9′	65.9	− 988	−15.0	− 1 208	−18.3
9 -10′	87.7	− 1 081	−12.3	− 1 363	−15.5
10 -11′	102.8	− 1 790	−17.3	− 2 274	−22.0
11 -12′	129.7	− 2 437	−18.8	− 3 123	−24.1

Member (see Figs. 14 and 15)	Area, in square inches	Initial Stage		Final Stage	
		Stress, in kips	Stress, in kips per square inch	Stress, in kips	Stress, in kips per square inch
Transverse Middle Brace; Carbon Steel					
10′ -20	165.0	+ 1 415	+ 8.5	+ 1 756	+10.6
20 -21	165.0	+ 1 170	+ 7.1	+ 1 506	+ 9.1
19M-21M	201.0	+ 684	+ 3.4	+ 845	+ 4.2
10′ -19M	129.0	+ 554	+ 4.3	+ 676	+ 5.2
9′ -19M	129.0	+ 557	+ 4.3	+ 764	+ 5.9
19M-20	78.8	− 329	− 4.2	− 415	− 5.3
20 -21M	39.0	+ 240	+ 6.2	+ 326	+ 8.4
19M-19	39.0	− 15	− 0.4	− 15	− 0.4
20M-20	39.0	+ 34	+ 0.8	+ 34	+ 0.8
21M-21	39.0	− 18	− 0.5	− 18	− 0.5
Transverse Bottom Brace; Carbon Steel					
4′ -25	179.0	+ 675	+ 3.8	+ 1 211	+ 6.8
26 -28	95.5	+ 438	+ 4.6	+ 605	+ 6.3
3′ -27M	107.8	+ 397	+ 3.7	+ 476	+ 4.4
27M-28M	107.8	+ 147	+ 1.4	+ 155	+ 1.4
3M-26	83.4	+ 407	+ 7.6	+ 598	+11.2
2M-29	69.0	− 568	− 8.2	− 783	−11.3
26 -27M	90.8	− 124	− 1.4	− 130	− 1.4
27M-28	119.2	− 608	− 5.1	− 829	− 6.9
29 -27M	119.2	− 781	− 6.5	− 1 078	− 9.0
27M-27	47.0	− 21	− 0.5	− 197	− 4.2
28 -25	64.4	− 406	− 6.3	− 406	− 6.3
28M-28	47.0	+ 42	+ 0.9	+ 42	+ 0.9
Longitudinal Diagonals in Plane of Inner Columns; Carbon Steel					
0′ - 1′	47.0	− 646	−13.8	− 865	−18.4
1′ - 2′	47.0	− 647	−13.8	− 872	−18.6
2′ - 3′	47.0	− 648	−13.8	− 879	−18.7
3′ - 4′	47.0	− 654	−13.9	− 895	−19.1
4′ - 5′	47.0	− 515	−11.0	− 577	−12.3
5′ - 6′	47.0	− 533	−11.3	− 592	−12.6
6′ - 7′	47.0	− 556	−11.8	− 613	−13.0
7′ - 8′	47.0	− 575	−12.2	− 623	−13.3
8′ - 9′	47.0	− 593	−12.6	− 646	−13.8
9′ -10′	94.0	− 1 003	−10.7	− 1 206	−12.8
10′ -11′	94.0	− 1 072	−11.4	− 1 243	−13.3
11′ -12′	6-1″ plates	− 979	−10.4	− 1 081	−11.5
Longitudinal Diagonals in Plane of Outer Columns; Carbon Steel					
0 - 1	47.0	− 575	−12.2	− 709	−15.1
1 - 2	47.0	− 575	−12.2	− 715	−15.2
2 - 3	47.0	− 560	−11.9	− 703	−15.0
3 - 4	47.0	− 559	−11.9	− 708	−15.1
4 - 5	47.0	− 538	−11.4	− 565	−12.0
5 - 6	47.0	− 537	−11.4	− 551	−11.7
6 - 7	47.0	− 543	−11.6	− 556	−11.8
7 - 8	47.0	− 530	−11.3	− 532	−11.3
8 - 9	47.0	− 526	−11.2	− 525	−11.2
9 -10	55.4	− 539	− 9.7	− 549	− 8.9
10 -11	47.0	− 390	− 8.3	− 426	− 9.1
11 -12	47.0	− 196	− 4.2	− 222	− 4.7

TABLE 4.—(*Continued*)

Member (see Figs. 14 and 15)	Area, in square inches	Initial Stage		Final Stage	
		Stress, in kips	Stress, in kips per square inch	Stress, in kips	Stress, in kips per square inch
Transverse Horizontals; Carbon Steel					
0 - 0′	71.0	0	0	0	0
1 - 1′	47.0	+ 539	+11.5	+ 631	+13.4
2 - 2′	47.0	+ 479	+10.2	+ 559	+11.9
3 - 3′	68.0	+ 725	+10.7	+ 870	+12.8
4 - 4′	82.4	+ 738	+ 9.0	+ 948	+11.5
5 - 5′	47.0	+ 471	+10.0	+ 566	+12.1
6 - 6′	47.0	+ 455	+ 9.7	+ 551	+11.7
7 - 7′	47.0	+ 457	+ 9.7	+ 558	+11.9
8 - 8′	47.0	+ 480	+10.2	+ 587	+12.5
9 - 9′	77.0	+ 617	+ 8.0	+ 761	+ 9.9
10 -10′	77.0	+ 971	+12.6	+ 1 211	+15.7
11 -11′	126.8	+ 878	+ 6.9	+ 1 110	+ 8.8
12 -12′	229.5	+ 1 017	+ 4.0	+ 1 279	+ 5.1
Transverse Top Brace; Carbon Steel					
12′ -14	304.0	— 2 380	— 7.8	— 2 920	— 9.6
14 -15	304.0	— 2 322	— 7.6	— 2 872	— 8.6
11′ -17	126.8	+ 587	+ 4.6	+ 693	+ 5.5
17 -18	126.8	— 533	— 4.2	— 630	— 5.0
12′ -17	78.8	— 735	— 9.4	— 923	—11.8
11′ -14	78.8	— 231	— 3.0	— 299	— 3.8
14 -15M	78.8	— 315	— 4.0	— 451	— 5.8
17 -15M	78.8	+ 315	+ 4.0	+ 451	+ 5.8
17 -14	47.8	— 172	— 3.7	— 222	— 4.7
10′ -17	96.3	— 1 097	—11.3	— 1 392	—14.4

Member (see Figs. 14 and 15)	Area, in square inches	Initial Stage		Final Stage	
		Stress, in kips	Stress, in kips per square inch	Stress, in kips	Stress, in kips per square inch
Longitudinal Horizontals in Plane of Inner Columns; Carbon Steel					
0′ - 0′	71.0	0	0	0	0
1′ - 1′	47.0	+ 368	+ 7.8	+ 457	+ 9.7
2′ - 2′	47.0	+ 370	+ 7.9	+ 462	+ 9.8
3′ - 3′	47.0	+ 379	+ 6.4	+ 475	+ 8.1
4′ - 4′	59.0	+ 378	+ 6.4	+ 477	+ 8.1
5′ - 5′	47.0	+ 375	+ 8.0	+ 475	+10.1
6′ - 6′	47.0	+ 380	+ 8.1	+ 484	+10.3
7′ - 7′	47.0	+ 384	+ 8.2	+ 489	+10.4
8′ - 8′	47.0	+ 388	+ 8.3	+ 495	+10.5
9′ - 9′	118.0	+ 586	+ 5.0	+ 750	+ 6.4
10′ -10′	118.0	+ 781	+ 6.6	+ 1 003	+ 8.5
11′ -11′	154.0	+ 686	+ 4.5	+ 885	+ 5.8
12′ -12′	6-1″ plates	+ 295	+....	+ 381	+....
Longitudinal Horizontals in Plane of Outer Columns; Carbon Steel					
0 - 0	71.0	0	0	0	0
1 - 1	47.0	+ 320	+ 6.8	+ 400	+ 8.5
2 - 2	47.0	+ 324	+ 6.9	+ 406	+ 8.6
3 - 3	53.0	+ 326	+ 6.2	+ 409	+ 7.7
4 - 4	59.0	+ 331	+ 5.6	+ 417	+ 7.1
5 - 5	47.0	+ 330	+ 7.0	+ 416	+ 8.8
6 - 6	47.0	+ 324	+ 6.9	+ 410	+ 8.7
7 - 7	47.0	+ 315	+ 6.7	+ 400	+ 8.5
8 - 8	47.0	+ 306	+ 6.5	+ 387	+ 8.2
9 - 9	59.0	+ 306	+ 5.2	+ 389	+ 6.6
10 -10	59.0	+ 257	+ 4.4	+ 327	+ 5.5
11 -11	77.0	+ 139	+ 1.8	+ 178	+ 2.3
12 -12	95.0	+ 38	+ 0.4	+ 48	+ 0.5

The sections of the towers were designed not to exceed the following unit stresses, in pounds per square inch (l = unsupported inches of column or flange; r = radius of gyration; and b = width of flange, in inches):

Silicon Steel:

Tension 27 000

Compression ..... $27\,000 - 80\frac{l}{r}$, max. 23 000

Bending ..... $27\,000 - 270\frac{l}{b}$, max. 23 000

Shear 17 000

Bearing 40 000

Carbon Steel:

Tension 20 000

Compression ..... $20\,000 - 60\frac{l}{r}$, max. 17 000

Bending ..... $20\,000 - 200\frac{l}{b}$, max. 17 000

Shear 12 500

Bearing 30 000

Power-driven rivets: Shear..... 12 500

Power-driven rivets: Bearing..... 25 000

For the final stage—that is, for a dead load of 39 000 lb. and a live load of 8 000 lb. per ft. of bridge—an increase of unit stress was allowed for columns of silicon steel to 28 000 lb., and for columns of carbon steel to 20 500 lb.

The stresses were not to be exceeded for the following two ruling conditions: (1) Dead load plus live load plus temperature, or dead load plus wind;

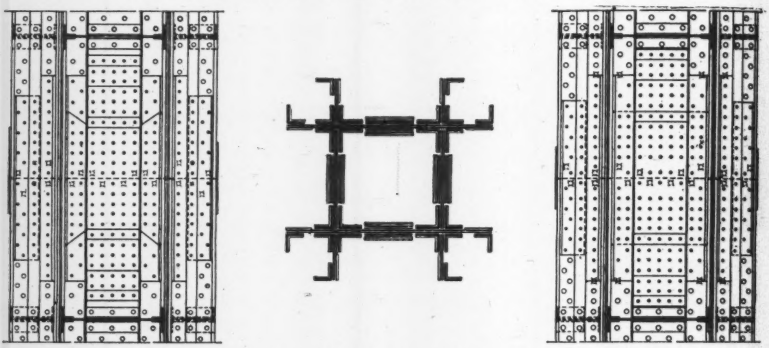


FIG. 16.—TYPICAL COLUMN SPLICE.

and (2) the ratio of the stresses based on the initial stage to those based on the final stage is approximately the same as the ratio between the two sets of unit stresses.

TABLE 5.—PHYSICAL AND CHEMICAL CHARACTERISTICS OF TOWER COLUMN STEEL
(1 kip = 1 000 lb.)

Item	Description	Carbon Steel			Silicon Steel	
		Specification		Test specimen	Specification	Test specimen
		Structural	Rivet			
(1)	(2)	(3)	(4)	(5)	(6)	(7)
	Chemical Properties (Maximum percentage):					
(1)	Carbon	.....	.....	0.21	0.40	0.35
(2)	Phosphorus, acid	0.06	0.04	0.018	0.06	0.022
(3)	Phosphorus, basic	0.04	0.04	.....	0.04	.....
(4)	Sulfur	0.05	0.045	0.037	0.05	0.037
(5)	Silicon	.....	0.45	.....	*	0.27
(6)	Manganese	.....	.....	0.50	.....	0.78
	Physical Properties:					
(7)	Tensile strength, in kips per square inch	58 to 68	52 to 60	63.6	80 to 95	88
(8)	Yield point (minimum) in kips per square inch	35	30	38.2	45	50.8
(9)	Elongation in 8 in. (minimum percentage)	†	†	28	†	22
(10)	Reduction of area (minimum percentage)	42	52	52	30	43
	Bend Test:‡					
(11)	Material, ¾ in. or less; bend 180°	Around $D = T$	Flat	.....	Around $D = T$	.....
(12)	Material more than ¾ in. and less than 1¼ in.; bend 180°	Around $D = 1.5\ T$	.....	.....	Around $D = 1.5\ T$	.....

* Not less than 0.20 per cent.

† Minimum elongation expressed by the ratio, $\frac{1\ 500}{\text{tensile strength, in kips}}$

‡ D = inside diameter of the bend; T = thickness of the material.

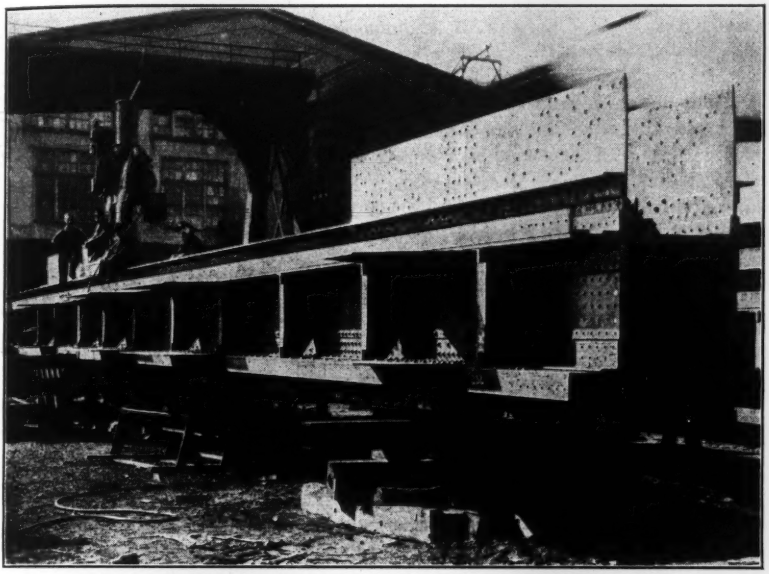


FIG. 17.—VIEW OF COLUMN RIVETED IN THE SHOP.

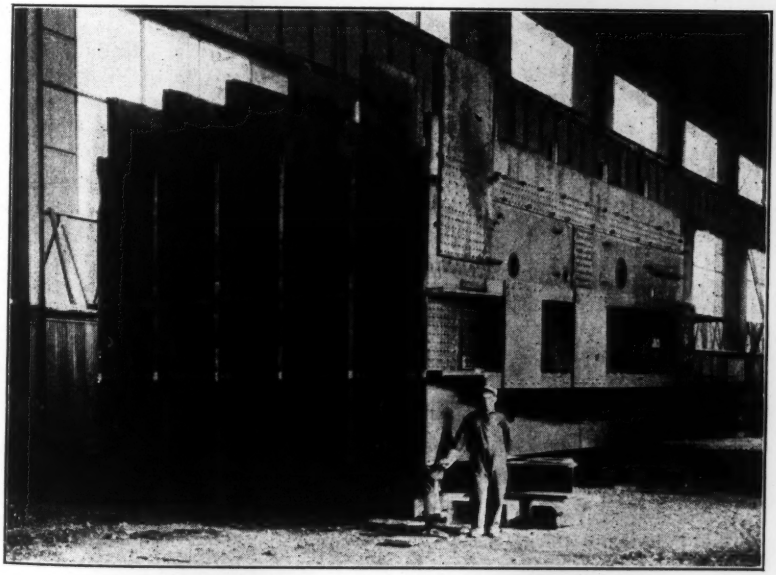


FIG. 18.—VIEW OF TOP SECTION OF INNER COLUMNS.

The column splices were developed for not less than 50% of the stresses for completely milled compression members in bearing. A typical column splice is shown in Fig. 16. Fig. 17 shows a column riveted in the shop and Fig. 18, the top section of the inner column.

It is gratifying to note here that the equivalent dead load for the final stage will be about 36 000 lb. per ft., leaving a margin for contingent loads of about 3 000 lb. per ft. against the design load of 39 000 lb.

The specified maximum chemical content of these grades of steel is given in Table 5, Items (1) to (6). The chemical and physical properties of the material actually used were determined by making 1 884 tests of 895 melts of silicon steel and 2 207 tests of 1 134 melts of carbon steel. The averages are given in Table 5, Columns (5) and (7).

Tests of Columns

In view of the unprecedented span of the bridge, the enormous forces the towers are called upon to sustain, and the great cost of the structure, it was considered important to ascertain by tests, as closely as practicable, the actual strength of the steel columns. While the results of relatively few tests made on large fabricated columns and particularly on silicon steel were available, it was considered advisable to complement the information and gain additional assurance and confidence in a structure of the magnitude of the George Washington Bridge. It was intended to determine the probable strength of the tower columns by testing other columns similar in design and dimensional proportions. Therefore, a program for such large-scale tests was prepared and carried out jointly by the Port of New York Authority and the National Bureau of Standards, in Washington, D. C.[3] R. S. Johnston, M. Am. Soc. C. E., Engineer of Research and Tests, The Port of New York Authority, was in charge of the tests. To this program models of the lower chord section of the Bayonne Bridge at Bayonne, N. J., were added later.

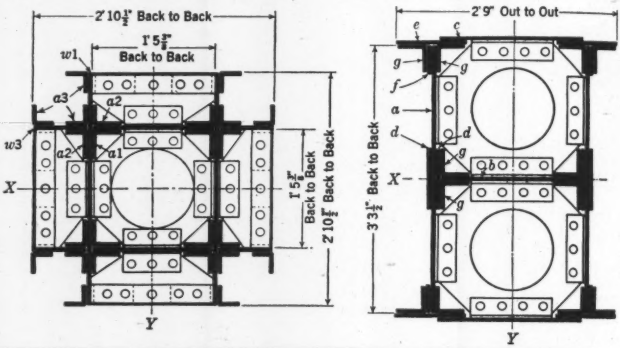
The capacity of the 5 000-ton testing machine at the Bureau of Standards, and the fact that the test columns should be similar in design and dimensionally proportioned to the tower-column section of the George Washington Bridge, determined the make-up of the test columns.

The column section selected for test purposes was approximately a half-size reduction of the typical tower column selected for study. The ratio of the respective areas of the test specimens and of the tower column was 1:4.5. The $\frac{l}{r}$ ratios were approximately equal, so that, in view of the relatively low value of $\frac{l}{r}$ of these columns, the fiber stresses due to bending, under loads proportional to their areas, were practically the same. The probable strength of the typical tower column section was thus directly proportional to that of the test column section on the basis of their respective sectional areas.

[3] A complete report of the column tests described herein is being prepared by the National Bureau of Standards.

Eight column sections, of three grades of structural steel, were tested—four with cross-sectional areas of 159 sq. in., two of 155 sq. in., and two of 151 sq. in. Six of these columns were dimensionally half-size specimens of the base columns of the towers of the George Washington Bridge. The remaining two sections were similarly proportioned models of a representative group of the lower chord sections of the Bayonne Bridge.

TABLE 6.—CHARACTERISTICS OF TYPICAL TEST COLUMNS



Subject	(a) COLUMN SECTION, GEORGE WASHINGTON BRIDGE				(b) TOWER CHORD SECTION, BAYONNE BRIDGE	
	Carbon steel and silicon steel columns		Carbon-manganese steel columns		Carbon-manganese steel columns	
	Symbol		Symbol		Symbol	
Main material	*W* 1	2 plates, 34⅜ in. by ⅝ in.	*W* 1	2 plates, 34⅜ in. by ⅞ in.	*a*	4 plates, 19¼ in. by ⅞ in.
	W 2	2 plates, 17 in. by ⅝ in.	*W* 2	2 plates, 17 in. by ⅞ in.	*b*	1 plate, 20¼ in. by ½ in.
	W 3	4 plates, 7⅝ in. by ⅝ in.	*W* 3	4 plates, 7⅝ in. by ⅞ in.	*c*	2 plates, 21 in. by ⅝ in.
	a 1	4 angles, 4 in. by 4 in. by ⅞ in.	*a* 1	4 angles, 4 in. by 4 in. by ⅞ in.	*d*	4 plates, 9 in. by ½ in.
	a 2	8 angles, 4 in. by 3 in. by ½ in.	*a* 2	8 angles, 4 in. by 3 in. by ½ in.	*e*	4 plates, 10 in. by ⅝ in.
	a 3	12 angles, 3 in. by 3 in. by ½ in.	*a* 3	12 angles, 3 in. by 3 in. by ½ in.	*f*	4 plates, 4 in. by ½ in.
					g	12 angles, 4 in. by 4 in. by ½ in.
Section area, in square inches.	.....	159	.....	151	.....	155
Length, in feet..	.....	24.00	.....	24.00	.....	20.75
Radius of gyration, in inches.....	Axis *X*–*X*	9.97	Axis *X*–*X*	9.97	Axis *X*–*X*	14.07
	Axis *Y*–*Y*	9.97	Axis *Y*–*Y*	9.97	Axis *Y*–*Y*	10.51
Slenderness ratio....	Axis *X*–*X*	28.90	Axis *X*–*X*	28.90	Axis *X*–*X*	17.70
	Axis *Y*–*Y*	28.90	Axis *Y*–*Y*	28.90	Axis *Y*–*Y*	23.70

The test columns were made of carbon steel, silicon steel, and carbon-manganese steel. The material was made to conform to the specifications previously given in this paper. The details of the column sections are shown in Table 6. The column section in Table 6(*a*) was dimensionally one-half the size of the tower column in Panel 0′-1′ of the George Washington Bridge, while that in Table 6(*b*) was one-half the size of the lower chord section, *L*18-*L*19, of the Kill van Kull (or Bayonne) Bridge.

The total compression in a gauge length of either 20 or 15 ft. was measured; so were the lateral deflection of the column and the possible tendency of the cross-section to change its geometrical shape under test. A secondary study was made of stress distribution at various points of the section by the use of telemeters.

The total compression for the tower columns was measured by sixteen dial compressometers, 20 ft. long, attached four to each column face, two on the edges of the outstanding flanges, and two to the inner corner angles connecting the outstanding flanges to the webs of the box section.

The columns were subjected to increment loading and observations were made for each additional increment. Instrument readings were continued until the columns had about reached their maximum carrying capacity, after which a series of load-time measurements were made to observe any pick-up. A summary of the results of the tests is given in Table 7. In this table the

TABLE 7.—SUMMARY OF TEST RESULTS, LARGE-SIZED COLUMNS
(1 kip = 1 000 lb.)

Item	Description	George Washington Bridge						Bayonne Bridge	
		Carbon		Silicon		Carbon Manganese		Carbon Manganese	
(1)	(2)	Test Column V (3)	Test Column VI (4)	Test Column VII (5)	Test Column VIII (6)	Test Column 1 (7)	Test Column 2 (8)	Test Column 1 (9)	Test Column 2 (10)
	Test Data (in Kips per Square Inch):								
1	Proportional limit	17.00	18.00	22.00	20.00	24.00	22.00	24.00	23.00
	Stress at Failure:								
2	First maximum	33.60	33.50	52.80	53.00	61.56	62.28	59.00	58.65
3	"Pick-up" maximum	36.80	36.70	55.80	54.90	.....	.....	.....	.....
	Yield Point Determined from Mill Test (in Kips per Square Inch):								
4	Average maximum	40.60	40.60	57.20	57.20	59.47	59.47	60.48	60.48
5	Average minimum	38.70	38.70	56.40	56.40	58.39	58.39	59.74	59.74
6	Weighted maximum	41.40	41.40	57.00	57.00	59.75	59.75	60.30	60.30
7	Weighted minimum	39.00	39.00	52.80	52.80	58.76	58.76	58.86	58.86
8	Percentage efficiency	83.6	83.3	96.2	96.5	103.9	105.1	99.0	98.4

values for the proportional limit (Item 1) were determined from the plotted test data showing the relation of applied stress to strain. The first maximum stress at failure (Item 2) is that at which failure is ordinarily considered to occur in column testing practice. The "pick-up" maximum stress (Item 3) is that which occurs after the point of first maximum stress, following long-continued loading with considerable deformation. The average maximum and minimum yield points (Items 4 and 5) were determined by averaging, respectively, the maximum and minimum yield-point values from coupon tests. The weighted maximum and minimum yield points (Items 6 and 7) were determined by weighting, respectively, the maximum and minimum values from coupon tests on the basis of the ratio of the area of the section (plate or angle) represented by the mill test, to the total section area of the test column. The efficiencies in Item 8, Table 7, are based on the mean of the weighted yield-point values (Items 6 and 7) and the first maximum stress

at failure (Item 2). These efficiencies express the ratio of the stress at which the column failed to that of the test coupon. The stress-strain curves developed by the test sections are shown in Fig. 19. The small slenderness ratio of the columns and their box construction did not develop any marked deflections before evidence had developed that the column was at the point of failure. A typical failure of a silicon steel column is shown in Fig. 20.

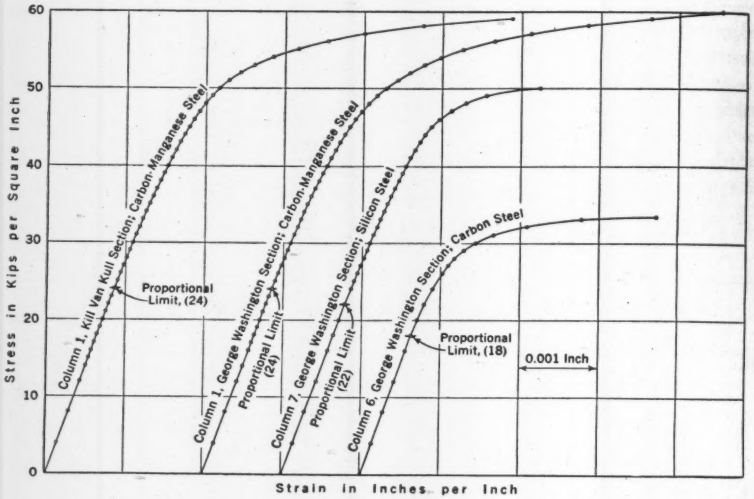


FIG. 19.—STRESS-STRAIN CURVES DEVELOPED BY TEST SECTIONS SHOWN IN TABLE 6.

From the results of the tests it has been found that the ratio of the strength of the test column to the mean average yield point of its material was 93.1 for the first maximum load, or 96.6 for the second maximum load. The probable stress, at failure, of the full-sized silicon steel tower column (Panel 0′-1′; area, 716 sq. in.), based on the foregoing observations, is 46 200 lb. per sq. in. for the first maximum load and 47 800 lb. for the second. Since the greatest allowable compressive stress on silicon steel will be 28 000 lb. per sq. in., this indicates a factor of safety for the full-sized column of 1.65 to 1.71, respectively.

To interpret the factor of safety correctly it should be kept in mind that the maximum compressive stress of 28 000 lb. is obtained under the reaction of the full dead and live load of the bridge, the dead load being 39 000 lb. and the live load, 8 000 lb. Should the live load be doubled, the stress in the columns will be increased by only 3 000 lb. per sq. in. To reach the ultimate strength of the towers the total load would have to be nearly doubled, which is an evident impossibility.

The results of this investigation on large-sized column sections confirmed the confidence of the engineers in the use of silicon steel and the box section for the tower columns of the bridge. They also furnished the Engi-

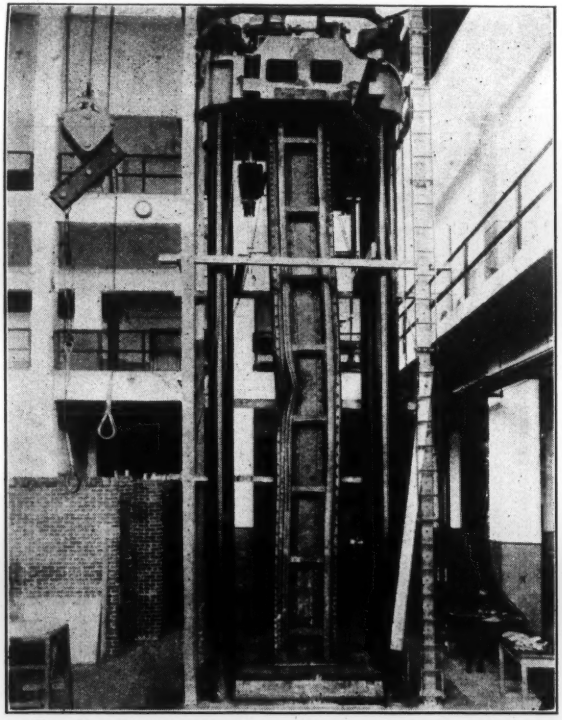


FIG. 20.—TYPICAL FAILURE, A SILICON STEEL TEST COLUMN.

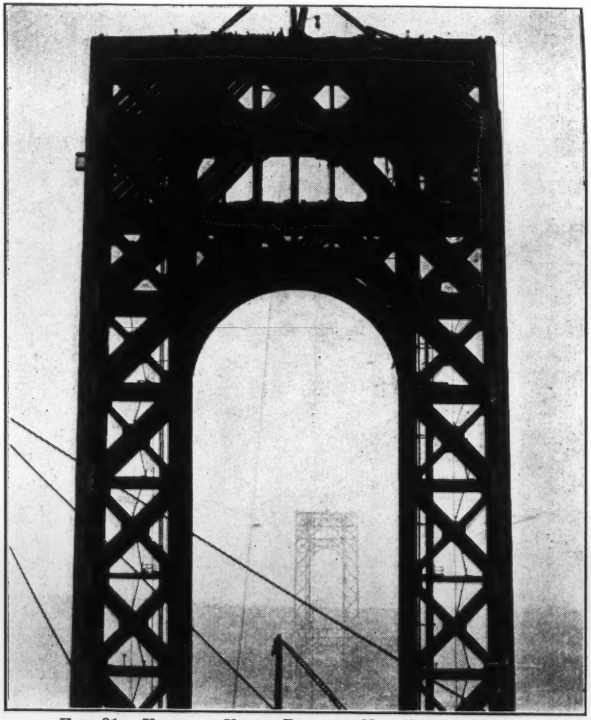


FIG. 21.—VIEW OF UPPER PART OF NEW JERSEY TOWER

neering Profession with information as to the behavior and strength of large fabricated columns and the efficiency of specific types of sections as compressive members. The direct comparison of three grades of structural steel on identical sections has been established by tests.

Tonnage and Cost

The completed towers contain 23 600 tons of silicon steel and 17 500 tons of carbon steel, 41 100 tons in all. The cost was an average of 9.7 cents per lb.

Conclusion

In this paper the reasons governing the adoption of the type of tower for the George Washington Bridge have been set forth. An interesting view of the top of the New Jersey tower is shown in Fig. 21. The analysis of the adopted type of an indeterminate tower and the method of procedure for the determination of the stresses have been described. It has been shown, furthermore, how the efficiency of this design has been tested in an analytical manner and further by stress-strain measurements on a model of a tower bent. Finally, the stress distribution in the tower columns has been determined by a series of careful stress-strain measurements on the completed tower under five progressive stages of loading. Furthermore, reference has been made to the tests of large-sized columns of make-up similar to that of the tower columns. In this way the process of the embodiment of the conception of the towers into the steel towers of the bridge has been related.

It was intended to present the story of the origin and development of the design conception of the towers, of the use of scientific methods of modern analysis of structures in the design, and of the application of methods of critical reasoning to its efficiency and stability. While supported by the engineering experience of the past, it was fully realized that mathematical analysis and reasoning are essentially mental operations and should be verified by observations of the physical behavior, in material embodiment. Observations were then made on a most readily available model of a bent. Finally, an extensive program of stress-strain measurements was carried out on the erected structure to verify the predictions of the engineering assumptions and analysis and to establish the efficiency of the behavior of the towers under load. To verify the behavior and the strength of the most important members of the towers, large-sized columns, about one-fourth the area, were tested to failure. The test results showed that the form adopted for the columns and their material and fabrication met the expectations of the designers.

It has been proved here again that the assumptions and methods of modern structural engineering are in close accord with the processes of Nature. It has given further assurance and confidence that engineering knowledge, the processes of mills and fabricators, and the skill of erectors can well be trusted to produce efficient and safe structures of this, and of greater, magnitude.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 1822

GEORGE WASHINGTON BRIDGE: CONSTRUCTION OF SUBSTRUCTURE

BY MONTGOMERY B. CASE,[1] M. AM. SOC. C. E.

SYNOPSIS

The principal features of the construction methods used on the foundations for the main piers and the anchorages for the cables of the George Washington Bridge are described in this paper. The foundation piers for the New Jersey tower involved the use of open coffer-dams, larger and deeper than any previously used on bridge construction. In New Jersey, the anchorage for the cables was obtained by excavating inclined tunnels in the solid rock of the Palisades in which the anchorage steel was embedded in concrete. The existence of Jeffreys Hook projecting into the Hudson River in the central part of Fort Washington Park provided an ideal site for the location of the piers for the tower, and the natural topography behind the point and adjacent to Riverside Drive offered adequate footing for the massive concrete anchorage at the New York end of the main bridge.

Preliminary studies had brought out clearly the many advantages of the location passing over the extreme point of Fort Washington Park. The narrowing of the river at this point made possible a shorter main span than at any other location in this vicinity. The ground on both sides of the river is high. On the New York side the sound rock forming Fort Washington Point offered excellent foundations for the tower, and the frequent outcroppings and exposed ledges at the site of the anchorage indicated that a rock foundation could be obtained under the entire area of the anchorage with relatively little excavation.

NEW JERSEY TOWER FOUNDATIONS

The preliminary borings for the foundation of the New Jersey tower had indicated that the piers could be founded on rock at a depth not to exceed

[1] Engr. of Constr., The Port of New York Authority, New York, N. Y.

100 ft. if they could be located near the shore line. While the pierhead line established by the War Department would have permitted the location of this bridge pier about 150 ft. farther into the stream, the studies indicated clearly that the difficulties and increased cost of carrying this pier to rock at the greater depth would more than offset any saving in the length of the main span.

The area of the base of the tower is so large that from the first it was apparent that, with the firm rock foundations available, it was not necessary to build a single large pier. The middle third could be omitted so that the foundation could take the form of a pier under each of the two legs of the tower.

The size and shape of the foundations were thus controlled to a considerable extent by the design of the tower. The adopted design provided a block of concrete under each tower leg as shown in Fig. 1. It is to be noted that the piers are faced with granite from the top to a point 7 ft. below mean low water.

The vertical load on the foundation amounts to an average of 22 tons per sq. ft. Under the most unfavorable combination of vertical and longitudinal forces the resultant will have an eccentricity of 5.2 ft., or only 5% of the width of the tower foundation. This will cause a maximum edge pressure on the rock foundations of 28 tons per sq. ft., or about 400 lb. per sq. in. without considering buoyancy.

Compression tests made of various pieces of cores from different rock strata indicate a variation in the strength of the rock of between 3 000 and 24 000 lb. per sq. in. and a probable average of the entire mass underlying the tower foundation of between 12 000 and 15 000 lb. per sq. in. The lowest test result obtained is, therefore, nearly eight times the maximum edge pressure and ten times the average pressure. The average strength of the rock is probably more than thirty times the edge pressure that will be transmitted by the mass concrete.

The early studies based on the preliminary borings contemplated that the pier would be sunk by the pneumatic process which, with the information then available, appeared to be the safest and most practicable method. However, the final borings indicated that the maximum depth to rock was only 75 ft. and the average depth less than 50 ft., which made it desirable to give consideration to the use of an open coffer-dam.

On account of the slope of the rock, if the pneumatic process had been used, it would have been desirable to divide the area of each pier into two or more caissons to avoid the necessity of excavating perfectly sound rock along the high side next to the shore line in order to carry the cutting edge within reach of the rock along the outboard or river edge of the caisson. The difficulty of obtaining effective connections between these caissons to insure a unified action under a single pier was recognized. This difficulty would not have to be met if the open coffer-dam was used, which method would allow a careful and reliable preparation of the rock bed upon which the foundation was to be placed.

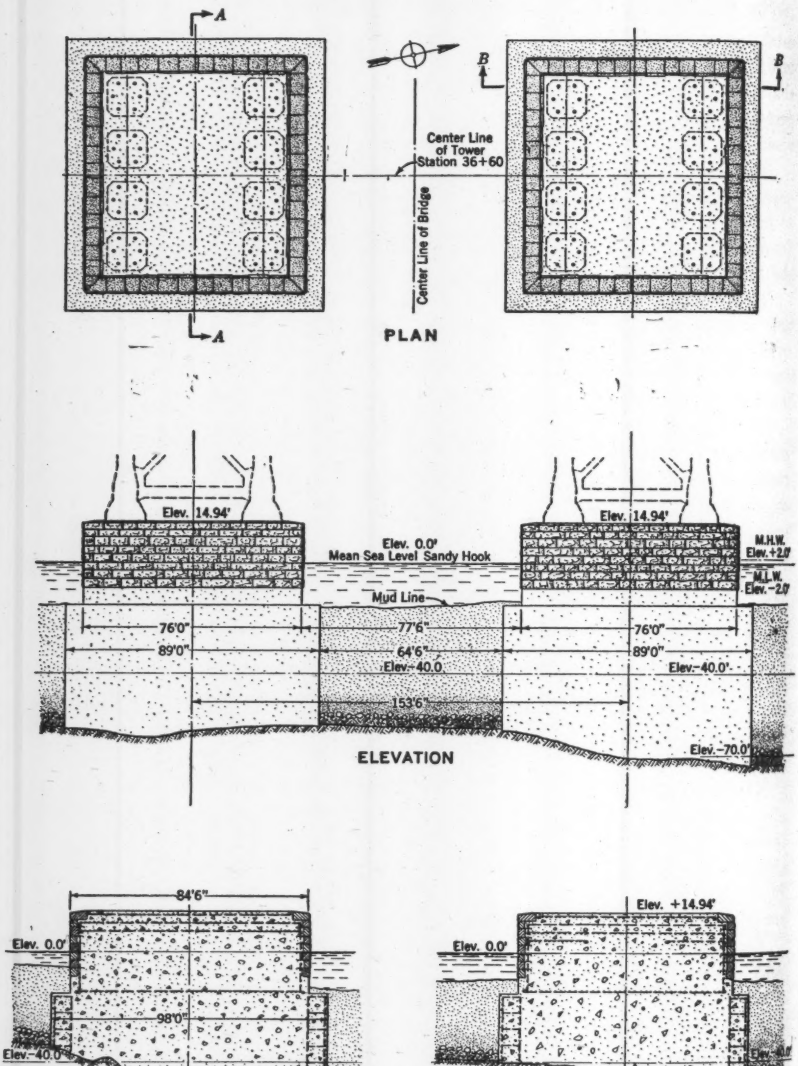


FIG. 1.—SECTIONS SHOWING DESIGN OF NEW JERSEY TOWER FOUNDATIONS.

The final borings had shown a fairly compact rock structure and the apparent absence of large seams or evidence of boulders which might have made the open coffer-dam method very difficult. Consequently, it was decided, upon the recommendation of Daniel E. Moran, M. Am. Soc. C. E., to invite alternate bids for these foundations based upon both the open coffer-dam and the pneumatic caisson methods. The design of the coffer-dams and the method of sinking them were optional with the contractor, subject to the approval of the Engineer.

The coffer-dam proposed by the contractor is shown on Fig. 2. It was to be built by first dredging the entire site of the two foundation blocks as a single excavation to a depth of 45 ft., at the river edge of the coffer-dam and to the rock surface over the remainder of the area. This would permit the assembling and sinking of timber bracing within the area of each coffer-dam previous to the driving of steel sheet-piling. This method would permit the bracing to be built down to within a satisfactory distance from the surface of the rock before any pumping was started and would avoid creating the condition in which the sheeting has considerable pressure to resist before the bottom sets of braces have been placed. The steel sheet-piling was to be driven in full lengths with the expectation that sufficient penetration could be obtained in the top layers of disintegrated rock to prevent any serious difficulty from inflow under the bottom of the sheeting.

The over-all dimensions of the interior of the coffer-dam were made 10 ft. greater in both directions than the dimensions of the pier so as to provide a 5-ft. clearance beyond the neat lines of the pier on all sides. Four horizontal frames and one inclined frame were provided in the original plan. The inclined frame was to be set at an angle approximately parallel to the slope of the rock floor as far down as the rock surface would permit, and was designed to resist the pressure on the outer or river face wall. Later, a second inclined frame, between the bottom frame and the lowest horizontal frame, was added to the design to reduce the stresses on the lower struts. Simple hydrostatic pressure was the basis for estimating the stresses, the influence of the silt and mud being disregarded. Each strut consisted of a pair of timbers, breaking joints and separated by the width of the vertical 12 by 12-in. posts, with 12-in. filler blocks at each joint which also served as splice material. The general arrangement is shown in Fig. 3, which is a view of some of the work on Contract HRB-2.[2] The pair of timbers in each strut varied in size from 6 by 12 in. at the top to 12 by 14 in. for the lower sets. At the intersections, vertical posts were framed between the timbers to provide a continuous member from the lower to the uppermost frame so that each frame could be built near the water surface and so that it could be sunk by the weight of the succeeding frame added above.

The rangers or walers were 24-in. and 30-in. I-beams, supported by timber brackets bolted to the under side of the strut and extending beyond the ends of the strut for the width of the beam. Where the steel walers overlapped at the corners they were secured by steel brackets or knee-braces bolted to

[2] See p. 84.

the inner flanges of the waler beams. Five struts were used longitudinally and five transversely, dividing each coffer-dam area into thirty-six pockets of about 16 by 18 ft. This arrangement provided unusually free working space for hoisting muck and lowering concrete. Diagonal timber bracing was used in the vertical plane from the top to the bottom set of each alternate row of struts to provide trussing to support the dead weight of the timber and such working platforms and miscellaneous loads as might be placed on the bracing.

A double wall of sheet-piling was used along the east or river face and inshore for about four-fifths of the distance on the up-stream and down-stream ends. For the remainder of the distance to the inshore wall, and along that wall, a single line of sheeting was used. The two lines of sheeting forming the double wall were spaced 7 ft. 10 in. apart and were connected by cross-diaphragm walls forming pockets or cells 10½ ft. long.

The dredging operation was started early in May, 1927, and more than 75 000 cu. yd. of the silt overlying the rock were removed in 20 days. Several large basalt boulders and an old rock-filled crib were encountered along the land side of the south coffer-dam which made it necessary to employ divers to drill and blast these obstructions. About 10 days were required for their removal.

After the dredging was completed, guide-piles were driven to support and control the sinking of the frames. These braced clusters of guide-piles were located so as to fall within the pockets formed by the bracing frames. As each frame was completed, it was lowered to such a level as to permit assembling the next frame just above the surface of the water, the vertical 12 by 12-in. posts serving to space the frames in the vertical position. As the lower frame approached the bottom of the dredged basin, further work by divers was required to clear up overlying riprap, boulders, and logs that interfered with the lowering of the bracing to its final elevation. The assembling and sinking of the frames were completed so that the sheet-pile driving could be started in July, 1927.

The steel sheet-piles weighed 32.9 lb. per ft. and varied in length from 40 ft. along the land side to 85 ft. along the deepest part of the river face. A total of 1 558 tons of sheeting was required for the two coffer-dams. The sheeting was set up against the steel waler of the bracing by means of a derrick-boat and driven with steam hammers both with and without swinging leads. (Fig. 3.) To avoid mistaking large boulders for the bed-rock, the final elevations of the points of the sheeting were checked carefully and compared with the elevation of the rock surface as determined by the final core borings. The greatest difficulty in seating the points of the piles against or into the rock was met in driving the sheeting on the shore side and on the ends on account of some interference from boulders and possibly some remains of the crib at the southwest corner of the south coffer-dam. The tendency of the tops of the longer piles to creep ahead also caused some difficulty which was overcome by providing a specially fabricated wedge-shaped pile which was inserted to bring the edges parallel at the position where the closing pile was placed.

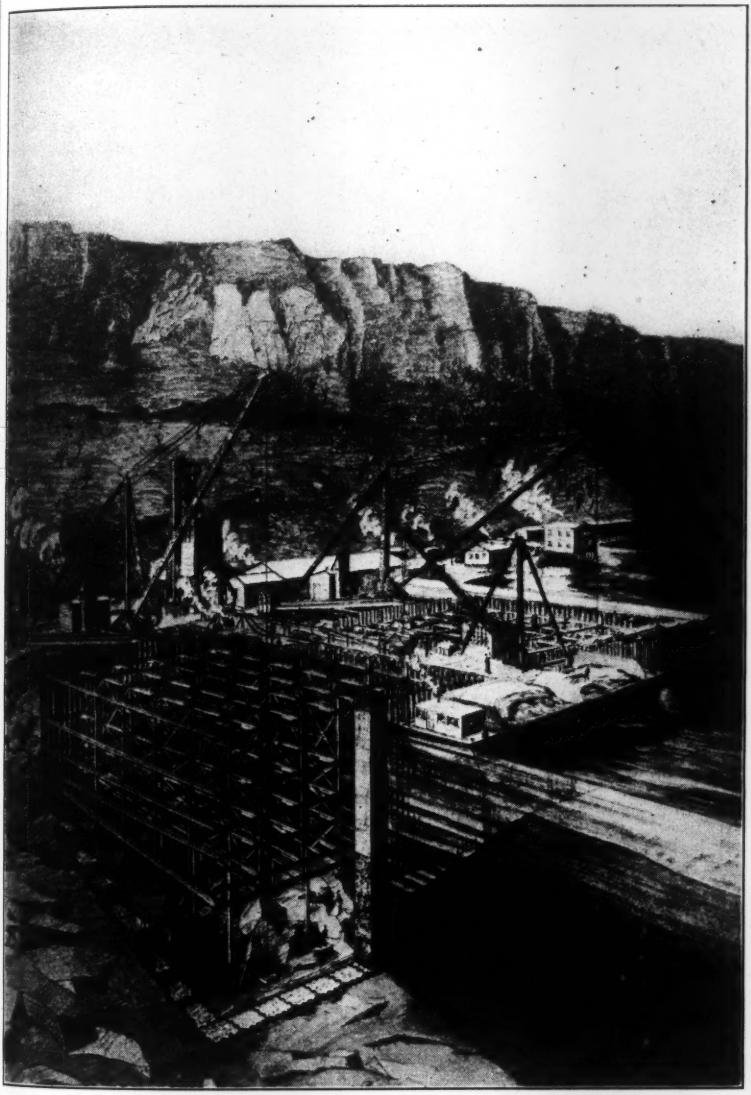


FIG. 2.—PERSPECTIVE OF COFFER-DAMS FOR NEW JERSEY TOWER FOUNDATIONS.

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FIG. 3.—TIMBER BRACING AND STEEL SHEET-PILING IN COFFER-DAMS, NEW JERSEY TOWER.

FIG. 4.—PREPARING FOUNDATION ROCK IN SOUTH COFFER-DAM ALONG WESTERLY WALL, NEW JERSEY TOWER.

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At first, all pockets formed by the double walls of the coffer-dam were to be filled with silt. However, in order to strengthen the wall below the bottom waler, concrete was used in the pockets along the river face and for a distance of four pockets on each end of the coffer-dam. This concrete extended from the rock surface up to the level of the lowest waler. To minimize the possibility that the exterior silt pressure might distort the vertical alignment of the sheeting, the silt on the inside was excavated in alternate pockets. The major portion of it was removed by a clam-shell bucket, but near the bottom, the remaining silt was taken out by suction on a centrifugal pump, guided by a diver to make certain that the rock surface was thoroughly cleaned of all silt and boulders and to insure close contact of the concrete seal in these pockets with the bed-rock. While the silt was being pumped, the water in the pockets was carefully maintained at a level, to avoid adding hydrostatic pressure to that of the exterior silt. Concrete was placed in these pockets with extreme care by a 1-yd., bottom-dump bucket, equipped with a canvas cover to prevent wash over the top surface as the bucket was lowered. After making certain that the bucket was on the bottom, it was raised slowly as it was dumped. In this manner the concrete was brought to an elevation slightly above the lower walers after which a mixture of river mud and sand was introduced as back-fill. The undredged pockets and the space behind the single pile wall along the land side were filled with similar material, as had been originally planned.

When the unwatering operation was begun, the precaution was taken to hold the water level just below each set of braces until all piles that were not bearing directly against the steel waler were blocked out, thus preventing distortion of the sheeting as the water was lowered. The coffer-dam was unwatered, without difficulty, by means of two vertical, submerged, electrically-driven centrifugal pumps, one an 8-in. and the other a 12-in. pump. When the water had been lowered to the level of the ground on the high side, the excavation of the remaining overlying material was begun. The softer materials were dug out with clam-shell buckets and the rough rock surface was cleaned by means of high-pressure water jets. Disintegrated rock was loosened with crow-bars and wedges and loaded by hand into hoisting skips. Little blasting was necessary.

The rock floor consisted of alternate layers of sandy shale and sandstone with bedding planes sloping toward the shore at an inclination of 15 to 20 degrees.[2] The foundation bed was prepared by removing the shale, leaving benches of the harder sandstone with vertical steps of several feet (Fig. 4). When the foundation rock along the shore side (amounting to about one-third of the total area) had been prepared, concrete was poured to an elevation sufficient to embed the lower two sets of bracing (Fig. 5). The concrete was carried beyond the neat lines of the pier into direct contact with the steel sheeting for a depth of 5 ft., to provide further security to the coffer-dam. This program was repeated around the ends and until the river wall of the coffer-dam had been sealed securely with concrete. The operations were then extended over the entire area, care being taken to stagger the construction

[2] See Fig. 12, p. 26.

joints. When the concrete had been poured to the elevation of the bottom of the granite facing, the coffer-dam was strutted against the concrete, and the set of braces next above was removed. As soon as the masonry had been built above high tide, the coffer-dam was flooded, the top set of bracing was removed, and the masonry was completed without obstructions. After the bottom had been sealed and while the masonry was being built up, these coffer-dams, more than 100 ft. square, were so tight that a 4-in. centrifugal pump, operating only part time, was sufficient to take care of all leakage.

When the foundation bed along the inshore edge of the north coffer-dam had been prepared and a portion of the concrete over this area had been placed, a blow-in occurred in a single line of sheeting at the up-stream, inshore corner, on the morning of December 23, 1927. The fissured rock failed to carry the reaction at the point of the sheeting, with the result that the sheeting was bent inward around the bottom waler as a fulcrum. Unfortunately, three men were killed; had the accident occurred at a later time in the day, the loss of life would have been much greater. The bent sheeting was removed, and this section was re-driven with a double wall. Since the rock foundation had already been prepared for concrete, it was possible to place a form under water and to remove the silt that had been carried in through the break. After this further preparation of the bottom by divers, sufficient concrete was placed by tremie in this corner to secure the toe of the new sheeting and to support the entire inshore, and part of the up-stream, wall of the coffer-dam (Fig. 6). The coffer-dam was then pumped out again and the work carried to completion, following substantially the same program. The rock along the outboard or river face of this coffer-dam was eroded to a level 78 ft. below high tide. Two additional walers were placed along this river face and strutted back to the rock.

All the construction plant on this contract (HRB-2) was electrically driven. Its layout is shown on Fig. 7. Concrete aggregate was delivered by water and unloaded on to a belt conveyor by a stiff-leg derrick, on a timber dock built between the two coffer-dams. The belt conveyor discharged into bins above a 2-yd. concrete mixer erected on shore. The batching was by weight and an automatic timing device controlled the minimum time of mixing. The cement was conveyed from the water-front on a belt supported by a trestle built along the up-stream end of the north coffer-dam. This belt, running back from the water-front, discharged on to a second belt running at right angles, which passed through the center line of a cement storage shed and up to the level of the mixer charging floor, so that cement was normally moved from the barge upon which it was delivered directly to the charging floor. When the concrete plant was not operating, the cement from the barge could be taken from the belt and stacked in the shed for storage, later to be loaded on the same belt for transportation to the mixer. The greater proportion of the cement was moved directly to the mixer without storage, resulting in considerable saving to the contractor. The mixed concrete was handled in 1-yd., bottom-dump buckets moved over a narrow-gauge track, in 2-car trains to within reach of the derricks that were located to serve each

FIG. 5.—CONCRETE ALONG SHORE SIDE OF SOUTH COFFER-DAM AND PREPARATION OF MIDDLE THIRD OF FOUNDATION, NEW JERSEY TOWER.

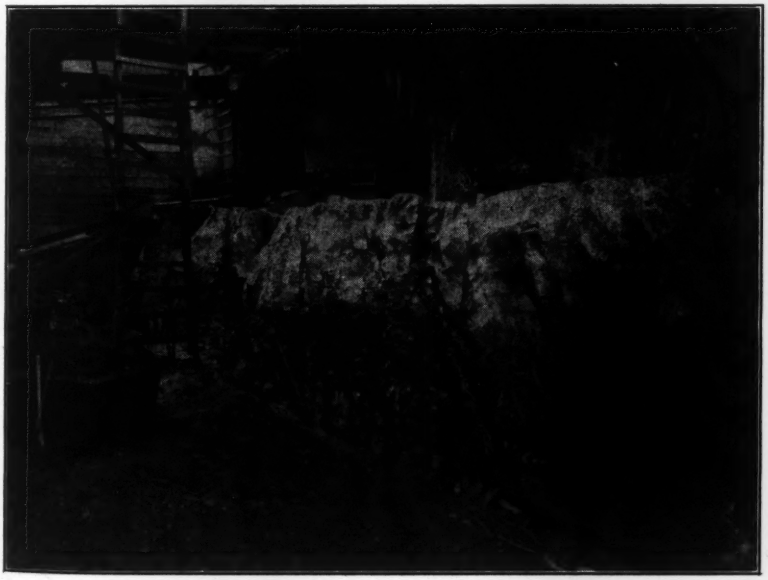

FIG. 6.—TREMIE CONCRETE AND FOUNDATION ROCK AT NORTHWEST CORNER OF NORTH COFFER-DAM, NEW JERSEY TOWER.

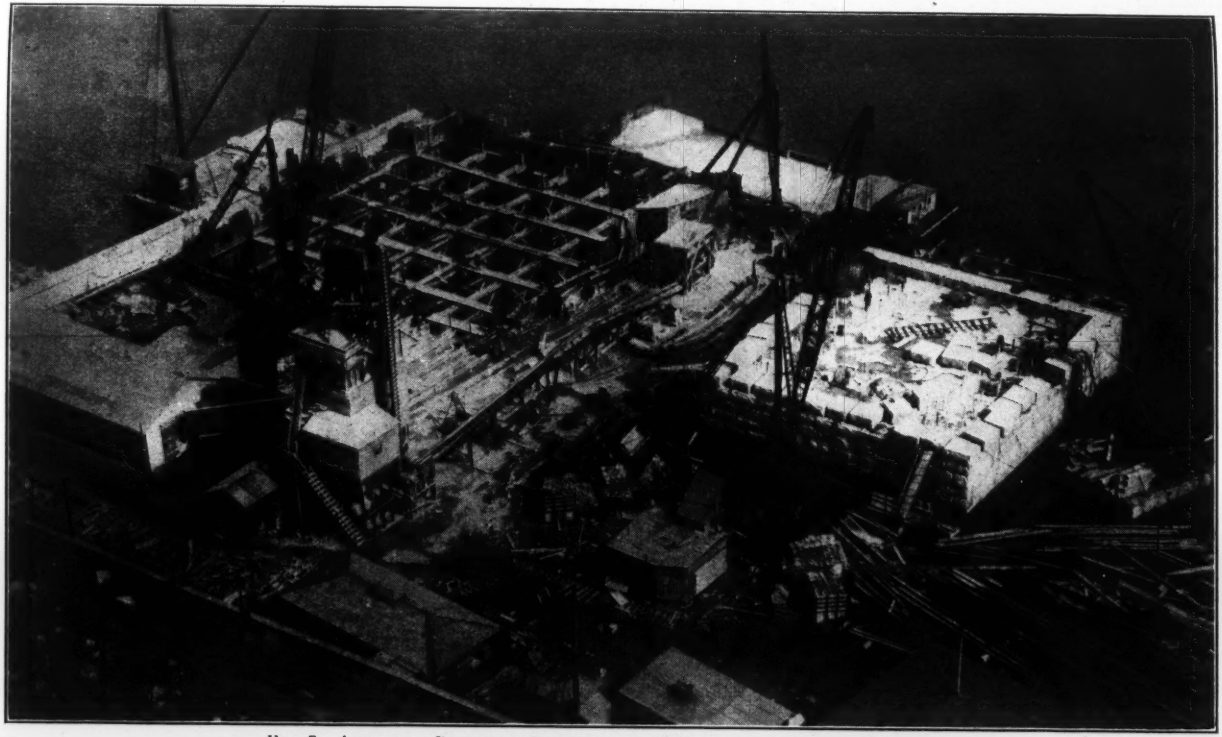

FIG. 7.—LAYOUT OF PLANT FOR CONSTRUCTION OF NEW JERSEY TOWER FOUNDATIONS.

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of the coffer-dams. These foundations were completed in April, 1928, and immediately afterward the erection of the bearings for the steel tower was begun. It is believed that open coffer-dams of any such area have not previously been carried to so great a depth.

New Jersey Anchorage

One of the conditions to be met in the construction of the New Jersey anchorage was to insure its completion in time for the wire-spinning operations on the main cables when the towers and the New York anchorage should be ready for this operation. Accordingly, the contract for the excavation of the New Jersey approach and anchorage tunnels (Contract HRB-3) provided that the work at the face of the Palisades and for a short distance back of the anchorage tunnels should be completed by October 1, 1928, while for the remaining excavation of the cut through the Palisades, completion was not required until January 1, 1930.

This anchorage consists essentially of two inclined tunnels or shafts, spaced 106 ft. apart, one for each of the two pairs of cables. In these inclined shafts the steel grillage girders and eye-bars to which the cable strands attach are embedded in concrete. These shafts were made wedge-shaped, with a cross-section of approximately 44 by 55 ft. at the bottom and 26 by 37 ft. at the top, the center line having an inclination of 37° 18′ from the horizontal. The anchorage girders are located 240 ft. along the axis of the tunnel below the turning point of the cables, which is a short distance below the upper roadway level.

The roof of the tunnel was excavated with an arch varying in radius from 25 to 30 ft. Provision was made in the contract for supporting this roof with timbering if it proved necessary. The contractor was to furnish and set whatever temporary timbering or shoring might be required for properly and safely carrying on the work. He was to be compensated on the basis of 50% of the actual cost of the material used and left in place after the excavation was completed. Although the roof span varied from 26 ft. at the mouth to 44 ft. at the bottom of the tunnel, it developed that no timbering of any kind was necessary.

Below the level of the future lower roadway, the excavation took the form of a rectangular pit, 47 ft. deep, 30 ft. wide, and 65 ft. long, which was necessary to provide sufficient working space during the operation of spinning the cable wire. The distance from the floor of the anchorage pits to the bottom of the tunnel excavation was 150 ft. along the axis of the tunnel.

The cable saddles at the New Jersey anchorage were supported by blocks of concrete, 26 ft. high and 27 by 69 ft. in plan, resting on the rock floor of the cut just forward of each of the anchorage pits (Fig. 8). These concrete masses also serve to anchor the lateral system in the horizontal plane of the wind chords which will function as the upper chords of the stiffening trusses when the lower deck is added to the existing structure. In the early design studies these anchorage saddles were supported on short rocker bents which would take care of the changes in cable length between the eye-bar heads and cable saddles. However, due to the close proximity of the anchorage for the

lateral system of the floor steel, it was found better to combine the support for these saddles and the lateral anchorage into a single concrete mass. These saddles also serve as splay castings, providing for the forces incidental to the splay of the sixty-one strands from the hexagonal shape at the saddles into the much larger rectangular shape where the individual strands pass around the strand shoes at the eye-bar heads and are connected to the eye-bars. Roller nests are provided to take care of the movements at this point. The center line of the cable changes direction by about 10° at the anchorage saddle, but the upper strands of the cable leading to the upper strand shoes have only a slight deflection downward. This was accomplished by locating these saddles so as to maintain a slight downward deflection of the upper strands. This insured their bearing in the saddles and avoided the disturbance of the strand adjustment that would result if the splay or deflection of the strands were made symmetrically about the center line of the cable, which would require that the upper strands be pulled down and restrained in the saddle after completion of spinning.

This concrete construction forming the support for the cable saddles of the anchorage was included in that part of the anchorage excavation contract that was to be completed by October 1, 1928, in order to avoid further division of responsibilty between the contractors and to insure that all parts of the New Jersey anchorage would be ready for the cable contractor in the fall of 1928.

In order to excavate the anchorage tunnels with the approach cut and anchorage pits, the contractor (Contract HRB-3) planned to excavate the tunnel from the bottom upward by adopting mining methods. A shaft located beyond the north rim of the cut and opposite the back or west end of the tunnels was sunk to a level somewhat below the lowest point of the tunnels (Fig. 9). From this shaft a cross-drift sufficiently wide to accommodate two mine tracks was driven to the southward to pass behind and slightly below the lowest point of the tunnels. This location was made such that the muck from the tunnels could be drawn off through chutes directly into the mine cars and thence hoisted up the shaft for disposal.

The anchorage tunnels and pits involved the excavation of 23 800 cu. yd. of basalt. The east or river end of the approach cut overlying the anchorage tunnels, which was to be completed by October 1, 1928, required the excavation of 99 600 cu. yd., while the remainder of the cut to be completed by January 1, 1930, involved a further excavation of 97 900 cu. yd. Before submitting his bid, the contractor had determined that advantageous arrangements for disposing of this 22 300 cu. yd. of excavated material could be made by crushing it at the site and selling it as crushed rock to dealers in concrete aggregates, withholding only sufficient quantity for the concrete to be placed in the saddle supports and anchorage tunnels. Accordingly, an efficient rock-crushing and screening plant was erected north of the rim of the cut through which the rock from the tunnels, pits, and approach cut was passed and delivered at the crusher bins to a local dealer who trucked it to a near-by site for storage.

FIG. 8.—NEW JERSEY ANCHORAGE BLOCKS IN OPEN CUT.

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It was the contractor's original intention to dig the tunnels by stoping methods, drawing off only sufficient muck to take care of the swell of the excavated material and leaving the remainder as a footing and support for the stoping drills which were to take out the full section of the tunnels as they progressed upward to the roof. It was found, however, that the slope of the tunnel was too great for the muck to remain in a stable pile on the floor and that it did not afford a sufficiently safe and secure support for the workmen and the drills. The procedure was changed accordingly after the

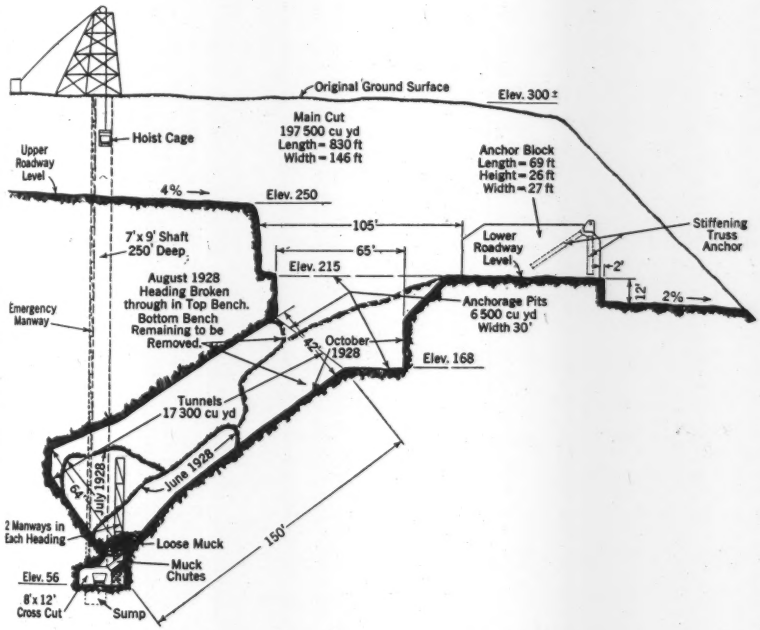

FIG. 9.—DIAGRAM OF ANCHORAGE TUNNEL EXCAVATION.

excavation for the full section had progressed about half-way up the length of the tunnels and from that point a full crown drift was taken out up to where the tunnel was holed through into the anchorage pits. The bench remaining below this crown drift was then shot, largely by rounds drilled down from the bench, some face drilling being done along the limiting planes of the tunnel. The program or schedule of drilling was varied to meet the shape, size, and position of that part of the heading to be blasted. In general, the shots included from 80 to 100 holes. The work on both tunnels progressed simultaneously, which made it possible to employ the muckers and drillers alternately in the other tunnel during the time necessary to ventilate the working chamber after each blast.

During the period when the full heading was taken out and the excess muck allowed to accumulate in the bottom of the tunnels, access to the working chamber was maintained by two timbered manways into each tunnel. These manways, in addition to the necessary ladders, also provided space for the air lines, water pipes, and the ventilating pipes. It was necessary, of course, to carry the drill steel, dynamite, and other supplies through these manways. Walkways were built to overcome some of the difficulties from the sliding of the muck and to provide more convenient access from the top of the manways to the headings. These were supported on transverse cables which were anchored into the side walls of the tunnel.

The handling of the long drill steel up through the manways and over the walkways to the heading proved an expensive and tedious procedure, so that after the heading had advanced a little more than one-third the way up the tunnel, an 8-in. vertical well drill hole was sunk from the bottom of the cut through to intersect each of the tunnels. The drill steel and dynamite could then be lowered into the tunnels from the surface by light, air-operated hoists. This greatly expedited the work and reduced the cost of transporting supplies to the working face. These 8-in. holes also supplemented the ventilation duct which was a 14-in. spiral riveted pipe leading down the hoisting shaft and up through the manways.

The survey lines and grade elevations in the tunnels required considerable tedious instrument work. Lines and grade elevations were projected down the hoisting shafts through the cross-drifts and up the manways into the tunnels. When the tunnels had been "holed through" into the anchorage pits, it was found that the instrument work checked within a few hundredths of a foot.

After the main excavation had been completed, the anchorage pits and tunnels were trimmed to clear all encroachments within the area required for the anchorage steel, and preparations were made for placing the anchorage girders and eye-bars. The relatively small amount of seepage into the pits and tunnels was drained to a sump at the foot of the hoisting shaft and pumped from that point. Rubble and concrete masonry was placed to seal off the openings from the bottom of the tunnels into the cross-drift, and concrete footings were placed for the support of the anchorage girders.

The accurate location and alignment of the anchorage girders and eye-bars was greatly facilitated by providing steel falsework which was embedded with the anchorage steel. This arrangement is well illustrated by Fig. 10, which is a view of the New York anchorage. This falsework was fabricated accurately and checked so that a minimum of instrument work in locating a few of the main members assured the correct location of all the anchorage steel. All the girders and the bottom three tiers of the eye-bars were put in place before any concrete was poured.

In order to reduce the effect of the clearance between the pin and the pin-holes in the eye-bars and girders, the eye-bars were held against the pin in the direction in which later the cable pull would be applied. This was accomplished by means of the U-bolts shown in Fig. 11, which passed around the

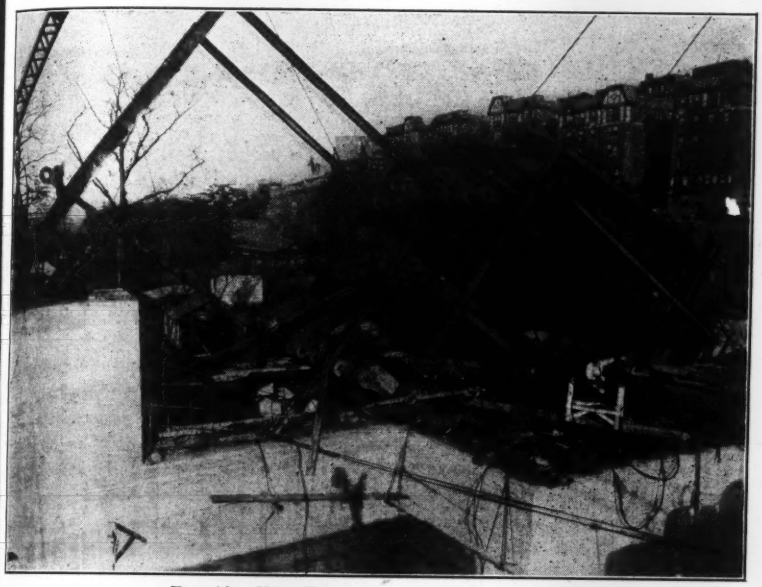

FIG. 10.—NEW YORK ANCHORAGE STEELWORK.

FIG. 11.—U-BOLTS AND COUPLINGS ON EYE-BARS.

head of each eye-bar and forced the head face of each pin-hole in close contact with the pin, thus eliminating the back lash that would occur if the bars were allowed to slack down so that the bar face of each pin-hole rested against the pin.

There were four lengths or tiers of eye-bars between the anchorage girders and the cable strand shoes. Only three tiers were placed and embedded in concrete previous to the wire-spinning operations. The eye-bars of the fourth tier were placed as the spinning progressed, thereby furnishing more working room during the spinning operation and making it possible to embed the upper or fourth tier of eye-bars after substantially all the dead load stress had been applied to them.

As a precaution against the shrinkage of concrete, which would permit ground-water to reach the anchorage steel, mats of reinforcing bars were provided on the six faces of the embedding concrete; that is, parallel to the side walls, roof, floor, and lower end and upper end of the anchorage tunnels. This reinforcement consisted of $\frac{3}{4}$-in. rods, spaced 12 in. in both directions. Construction joints in the embedding concrete were avoided by laying out a program that permitted concrete to be poured continuously.

Stop-waters were provided across the construction joint between the concrete embedding the lower three lengths of eye-bars and that placed about two years later around the upper eye-bars carrying the dead load stress of the erected bridge. These stop-waters consisted of a line of $\frac{1}{2}$-in. steel plates around the four faces of the anchorage mass, having one-half their width embedded in the original concrete and the other half projecting so as to be embedded in the final concrete. During the cable-spinning operation, the anchorage pits were drained by pumping from the construction shaft, into which the seepage flowed through pipes leading down through the embedding concrete into the cross-drift and shaft.

The lining of the anchorage pits was omitted until after all the spinning operations had been completed and the suspended structure erected, but before the concrete deck had been built over the anchorage area. Then it was anchored to the rock walls by 1-in. deformed bars spaced 5 ft. in each direction and anchored into holes drilled 3 ft. into the rock. The walls themselves were reinforced near the face with a mat of $\frac{3}{4}$-in. rods, spaced 12 in. in both directions. In addition, blind drains were provided along the water-bearing seams and brought through openings at the bottom of the wall to reduce the building up of hydrostatic pressure back of these lining walls. After the pits had been lined with concrete, the drainage was cut off at the bottom of the anchorage pits and the small quantity of seepage water was removed from the bottom of these pits into a drainage system at the level of the lower deck by sump pumps equipped with automatic float switches. Although the bottom of the anchorage pits is about 125 ft. below the general surface of the top of the Palisades, it has been found that a very small quantity of seepage water comes in through the lining. A heavy concrete cap was placed over the top of the construction shaft, but the cross-drift and shaft were not back-filled.

New York Tower Foundations

The site of the New York tower foundation is on Fort Washington Point opposite 178th Street, in Manhattan. Here, the typical Manhattan schist rises to an average elevation of 10 to 12 ft. above mean high water, with only two small inlets where the rock surface falls below the level of high tide. The preliminary borings showed sound rock, but persistent reports, based on soundings, that this rocky promontory was undercut by the river, and the sharp slope of the rock to a considerable depth, made additional exploratory core borings desirable. These further borings clearly indicated a distinct fault in the rock structure about 200 ft. east of the site, which was apparent from the valley behind the knoll forming the promontory. However, there was no indication of any objectional rock structure under the site, and inclined holes down the river slope proved to be in sound rock for a considerable distance on the river side of the pier. The elevation of the seams in the rock cores showed that the bedding planes sloped toward the river.

The elevation of the top of the pier is the same as on the New Jersey side, and provisions were made for the same dimensions of the finished pier. However, because of certain delays that occurred in procuring the right of way in Fort Washington Park, it was decided to omit the granite facing temporarily, in order to save the time necessary for selecting, quarrying, and cutting the granite, and thereby to advance the date on which the erection of the steel tower could be started in the spring of 1928. The granite facing on this pier was placed in the fall of 1932 and a 7-ft. iron picket fence was built around the top of the coping to prevent trespassing on the structure.

The New York tower foundation, therefore, consisted of two rectangular concrete piers each approximately 76 ft. by 84 ft. 6 in. at the tower base, with footings 80 ft. by 88 ft. 6 in., to include the granite facing. The top surface was reinforced with two mats of 1-in. square rods spaced at varying distances from 10 to 18 in., center to center, in both directions, and a single mat of similar spacing on each face. The faces were built with 2-in. horizontal offsets spaced 2 ft. apart. Hooked 1-in. rods, spaced 2 ft. 8 in. vertically and 10 to 18 in. horizontally, were used to provide a bond between the concrete backing and the granite.

The plans called for the rock to be excavated to Elevation — 11 in the south pier and to Elevation — 6 in the north pier, which plane it was believed would pass well below any surface seams and would provide an ample berm or shoulder of rock along the river face of the pier. The inlets, where the proposed excavation was below the river surface, were readily cut off by a short section of wooden sheet-piling along the south face of the south pier and by a rock dike backed with clay or silt along the north face of the north pier. When the excavation was well advanced, a clay-filled seam extending under substantially the entire area of the south pier was encountered. This seam was from a few inches in thickness to scarcely more than a bedding plane. The slope of the bedding planes toward the river was also somewhat more than had been expected from the core borings. It was accordingly determined to remove the rock to a considerably greater depth. This judg-

ment and decisions in all similar cases was passed upon by Charles P. Berkey, M. Am. Soc. C. E. The final excavation was extended down into sound rock at an average elevation of — 15 in the south pier and to an average elevation of — 5 in the north pier. The quality of the rock may be judged by reference to Fig. 12.

In order to insure the completion of these foundations in time for the tower erection at the earliest practicable date the concrete was mixed in a plant located on a barge moored alongside the site. Since the quantity of concrete involved was relatively small as compared with the anchorage mass, this made no material difference in the cost to the contractor and enabled him to have ample time to set up the more elaborate plant used for handling and mixing the concrete that went into the anchorage.

New York Anchorage

The New York anchorage site is in Fort Washington Park just west of Riverside Drive, opposite the city block between 178th and 179th Streets, Manhattan. The location of Riverside Drive placed certain limitations upon the length of the New York side span. To have located the anchorage east of Riverside Drive would have resulted in a side span slightly longer than desirable or compatible with the design of the towers. The longer side span would have subjected the tower to considerable more deflection under a varying live load than is the case with the shorter side span. The selection of the site for the anchorage west of Riverside Drive resulted in a somewhat shorter side span than has been customary in the proportioning of long-span suspension bridges; but it had certain advantages, which have been clearly stated in the paper by O. H. Ammann, M. Am. Soc. C. E.[4] The outcropping ledges of rock (see Fig. 13) and the preliminary borings supported the earlier decision that suitable foundations for the anchorage structure were available without excessive stripping of overburden or excavation of rock.

The anchorage structure as designed consists of a mass of concrete approximately 200 ft. in the dimension transverse to the bridge axis and 300 ft. in the dimension along the bridge axis, rising to a height of 85 ft. above Riverside Drive and approximately 120 ft. above the low point in the rock, which was about midway of the length of the anchorage. The anchorage is not a solid rectangular mass; only the rear or easterly third is practically solid with the two wings extending to the west, the extreme ends of which constitute the buttresses for the support of the anchorage cable saddles.

Because of the great dimensions of this mass, with its resulting shrinking during the process of cooling and setting, three expansion keyways were provided; one longitudinal to the center line dividing the structure into north and south halves and the other two transverse to the center line and located approximately at the two-third's point forward from the rear end of the structure (see Figs. 14 and 15). These keyways, 7 ft. wide and extending from the foundation rock to the top of the structure, were left open above Elevation 100 until all the concrete had been placed. The longitudinal

[4] See p. 46.

keyway was filled upon the completion of the concreting operations. The transverse keyways were left open until the dead load stresses resulting from the cables, floor system, and concrete deck, were in effect so that the deformations along the axis of the bridge could take place before the keyway was closed.

The concrete that embedded the upper tiers or links of eye-bars was also omitted until the dead load was acting on the cables. This concrete and that for filling the two transverse keyways were placed by the contractor for the roadway pavement after all that pavement had been put in place.

The anchorage girders embedded in this mass extend about 25 ft. below the rock line. In the early studies and on later drawings for this anchorage structure, provision was made for certain drainage tunnels to be excavated into the rock on the high side of the anchorage girder pit excavation with the idea of intercepting at least a portion of the ground-water and thereby reducing the water pressure against the embedding concrete. A more intimate knowledge of the rock structure at this particular site, as developed by the final borings, made it evident that thoroughly satisfactory results would be obtained by driving a single drainage tunnel through along the low point in the rock, which happened to be located at the transverse center line of the structure. Fortunately, the rock floor continued at a low elevation to the south, so that it was possible to drain this tunnel by gravity without rock excavation, thereby materially lowering the ground-water pressure on the entire mass.

The overburden and the disintegrated mica schist that had to be stripped seldom exceeded more than 2 to 6 ft., and along the line of the anchorage girder pits the rock was found to be unusually sound, closed-grained, and free from water-bearing seams, so that the wisdom of the decision to omit the drainage tunnels was substantiated thoroughly by the conditions encountered during the excavation. In the northwest corner, however, it was found that the rock was badly disintegrated and since the buttresses at the westerly end of the anchorage carry the thrust from the anchorage cable saddles, it was necessary to excavate somewhat deeper than had been contemplated at the time the contract drawings were prepared.

The anchorage structure is without reinforcement except at the surface, along all exterior faces, where it consists of 1-in. square bars on 18-in. centers both ways. On horizontal surfaces and in certain parts of the structure somewhat more highly stressed, the bars were placed on 12-in. centers. In addition to this surface reinforcement, all the exterior faces were studded with hooked 1-in. square rods, spaced about 4 ft. each way. These rods provide for bond between the concrete and the granite face that is to be added at a later date. A reinforcing mat was also provided in the surfaces in contact with the rock in the anchorage girder pits to give additional protection against shrinkage cracks in the concrete, which might allow the ground-water to reach the anchorage girders. The precaution was taken of pouring this concrete lining of the anchorage girder pits as a monolith. A grillage of rods was also placed beneath the anchorage cable saddles.

FIG. 12.—NEW YORK TOWER FOUNDATION LOOKING NORTHWEST IN SOUTH PIER AREA.

FIG. 13.—ROCK AT SITE OF NEW YORK ANCHORAGE.

FIG. 14.—EMBEDDING STEELWORK IN NEW YORK ANCHORAGE.

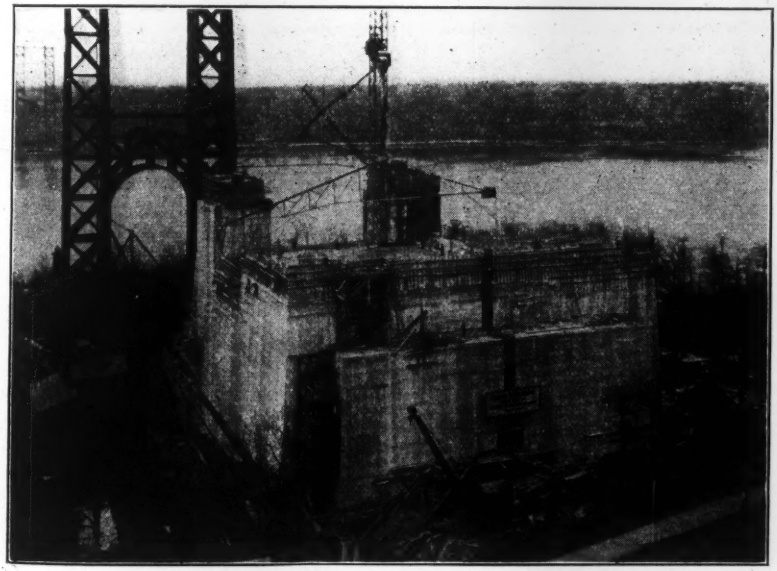

FIG. 15.—NEW YORK ANCHORAGE APPROACHING COMPLETION.

Recognizing the time element involved in obtaining the eye-bars and anchorage girders for this structure, provision was made in the contract for the cables, for that contractor to deliver the anchorage steel at the site without expense to the anchorage contractor. Provision was also made in the design for the steel falsework to support the anchorage material previous to its embedment (Fig. 10). As described for the New Jersey anchorage there was also a requirement that the eye-bars be held against the pin as shown in Fig. 11.

The concrete for the anchorage was designed for a compressive strength of 2 000 lb. per sq. in. in 28 days and was proportioned by the water-cement ratio method (as was the practice on the other foundation structures), with thoroughly modern weighing batchers.

The 110 000 cu. yd. of concrete involved in this structure justified a careful study of the plant to be selected for its accomplishment, and no doubt the successful bidder on Contract HRB-4 was aided materially in the prices he was able to bid by his confidence in the belt-conveyor plant that he adopted. In general, the writer understood that at the completion of this contract the cost of moving the cement and concrete aggregate by belt conveyor from the dock about 1 000 ft. distant to the structure was less than the cost of unloading the material from the barges to the dock. Because such belt-conveyor plants for moving the cement and aggregates from the dock to the structure and the mixed concrete from the central mixing plant to the foot of the hoisting tower were at that time somewhat unique, additional space will be devoted to a more detailed description of the plant used in constructing this anchorage.

The plant designed for transporting, mixing, and placing the concrete in the New York anchorage produced at times as much as 1 200 cu. yd. in a 16-hour day. The entire mass, approximately 110 000 cu. yd., was put in place in 5½ months, or somewhat more than 2 months ahead of the schedule. This was accomplished by a modern, electrically-operated mixing plant on the south side of the anchorage, approximately opposite the transverse center line. Fig. 16 shows the plant layout.

Sand and gravel were unloaded from barges by stiff-leg derricks and placed in separate piles over a 6 by 6-ft. wooden tunnel, 300 ft. long, which paralleled the dock. Hopper gates in the roof of the tunnel under the piles of aggregates allowed the latter to be fed to a belt conveyor in the tunnel. Sand and gravel were alternately dumped upon this belt as needed. The belt through the tunnel discharged directly upon a pair of belts in series which carried the aggregates up the incline of Fort Washington Park to the concrete plant. An interesting feature of the belts is the comparatively steep grade negotiated, which reached a maximum of 32% over a part of the length. This system had a capacity of more than 300 tons per hour and discharged into a 300-ton storage bin at the top of the mixing plant.

Cement was handled in bags directly from barges to the dock by cranes on the barges. A continuous belt from the dock to the mixing plant was located directly under the second and third sand and gravel belt. The bags

were placed on this belt by hand. As will be observed from Fig. 16 the belt passed through a cement storage house near the water-front. Thus, the cement could be taken off for storage or carried directly to the mixing plant as required. The cement belt had a capacity of 50 tons per hour and discharged on to a covered platform at the charging level of the mixing plant. The overhead bins at the mixing plant had a capacity of 100 tons of sand and

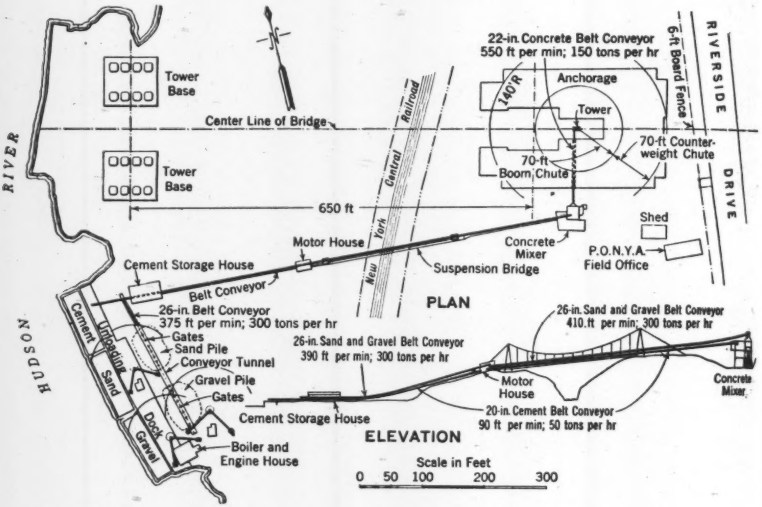

FIG. 16.—PLANT LAYOUT FOR TRANSPORTING, MIXING, AND PLACING CONCRETE, NEW YORK ANCHORAGE.

200 tons of gravel. Under normal operation the cement was delivered to the charging pipes at the rate of using in order to avoid re-handling. Only the excess due to short shot-downs was piled. A storage of 2 000 bags was provided on the covered platform at the charging level.

Two 1-yd., electrically operated, automatic time mixers were charged by a single weighing batcher plant. These mixers dumped the concrete into a common receiving hopper which, in turn, discharged to a belt conveyor, 125 ft. long, operating through the temporary tunnel through the south wing of the anchorage structure. This belt discharged into a receiving hopper at the base of a 260-ft. tower at the center of the anchorage.

The steel hoisting tower was equipped with two chuting systems, each consisting of a 75-ft. boom chute, from which was suspended a 75-ft. counterweight chute. In order that the desired low-slump concrete could be transported without clogging, 14-in. chutes were used to provide for easy flow. Concrete was hoisted to the receiving hoppers at the top of the boom chutes by a 36-cu. ft. roller hoist bucket, arranged to be dumped into either chute. This bucket traveled at a speed of 380 ft. per min., and was operated by a hoist driven by a 100-h. p. electric motor. The same hoist could also be used

to handle the chute booms and the lifting frames of the chute system when re-adjustments in level were being made. The discharge end of the counterweight chute was provided with a small hopper and a short "elephant trunk" to insure that concrete was always discharged in a vertical direction to avoid the segregation which would follow if it was allowed to be discharged at any angle much from the vertical. The flexibility of the chuting system was such that practically all the concrete could be discharged into its final position with almost no rehandling by hand.

In this large mass most of the concrete was placed in about 4-ft. layers and with the consistencies used, practically no tamping was required. With the large areas of form available for depositing concrete, it was seldom necessary to allow the concrete poured to be brought up more than 4 ft. at a time. Greater depth would invariably cause the excess of water to be "squeezed out"; it is quite impossible to avoid this increase in water content entirely no matter how low the slump is maintained.

Concrete Proportioning and Mixing

The specifications for all contracts on the George Washington Bridge calling for concrete, provided for the use of the cement-water ratio method in proportioning the mixtures. The strength of the concrete desired was named in the specifications with the further provision that,

"The proportions of the mixture of fine aggregate and coarse aggregate shall be such that generally speaking, the quantity of coarse aggregate shall not be less than the fine aggregate nor more than twice the fine aggregate."

It was required that the contractor should advise the Engineer of the source of the materials he proposed to use at least thirty-one days before the first concrete was to be placed. Samples were taken and tests were made, from which it was determined what the mixture should be, the cement-water ratio having been stated in the specifications.

From time to time throughout the work under each contract, samples were taken from the concrete, both at the mixer and from the forms, and made into cylinders for tests of the actual strength and densities obtained. Adjustments in the mixtures were made from time to time based on the variations in the fineness modulus of the concrete aggregates. The water content of the sand was checked from time to time in order to maintain control of the total water entering each batch. The test cylinders used were 8 by 16 in. made by the Field Staff of the Construction Division, in accordance with the standard methods of the American Society for Testing Materials for making and storing concrete specimens in the field (Serial Designation, *C*31-21). The contractor was required to supply adequate space and facilities at or near the mixing plant for making the cylinders. Occasional check tests were made by the Field Staff of the Port Authority's laboratory in Jersey City, N. J.

Contracts involving concrete provided for the furnishing of the cement by the contractor to the Port Authority at the contractor's bid price per barrel, so that the Port Authority retained complete freedom of judgment as to any variations in the mixtures that might prove desirable. The equity as between the Port Authority and the contractor is of course only maintained

within the reasonable limits commonly experienced with the aggregates available in the commercial markets tributary to the bridge construction.

It was provided that the consistency or workability of the mix should be measured by means of the slump test made in accordance with the tentative specifications of the American Society for Testing Materials (Serial Designation, *D*138-26*T*). The ordinary slumps required were between 2 and 3 in., but, of course, there were cases in which departures were made from these normal consistencies. As an explanatory phrase to the contractors it was stated in the specifications that if, at any time, the contractor desired a more workable mix than that obtained under the 3-in. slump, he must obtain the result by reducing the total quantity of aggregate per batch and not by the addition or diminution of water.

The specifications provided in general that all concrete should be produced in a central mixing plant and that previous to the beginning of the work, the contractor must secure the approval of the Engineer for the methods and apparatus he proposed to use. Except for the New Jersey anchorage all the concrete mixing plants on the foundations that are the subject of this paper, were provided with weighing batchers and time-control devices on the discharge lever of the mixer. For concrete used in the approaches to the bridge and for the New Jersey anchorage pre-weighed batches were hauled to the job in batch compartment trucks and mixed in standard paving mixers. In a few cases, for isolated operations at times when some of the main plants were not in operation, ready mixed concrete was used. Little difficulty was experienced in getting satisfactory results with the ready mixed product, but the precaution was taken to maintain an inspector at the plant during the time that the product was being delivered to the bridge.

In the case of the concrete used in the tunnels of the New Jersey anchorage a brief study was given to the possibilities of getting a dense, impervious concrete without spending too much for strength. There was no escape, however, from the generally accepted principle that imperviousness and strength follow along substantially parallel curves, so that considerable strength was built into the concrete in the New Jersey anchorage for the sole purpose of obtaining an impervious concrete where the strength was not essential.

In the case of the concrete used on the deck of the main bridge, the aggregates were hauled to the job in bulk, batched at the New York anchorage, and hoisted to the deck level. The dry and unmixed aggregates were then transported over the deck to the mixer alongside the point of deposit, so that the mixed concrete could be discharged directly into place in the deck slab. This was done in order to maintain a more precise control of the water content, which was so vitally necessary in producing a pavement of satisfactory quality.

The strengths obtained on the concrete used in the four principal foundations described in this paper are shown in Table 1.

In general, the results obtained on the various contracts for the George Washington Bridge fully demonstrate the economies possible by the use of the cement-water-ratio method in proportioning concrete. There were a few

instances, however, which demonstrate most forcibly the dangers involved where some unthinking individual attempts to follow the formulas blindly when the results clearly indicate that some blunder in the computations has been made. The writer is thoroughly convinced that still further economies

TABLE 1.—STRENGTHS OF TEST CYLINDERS, FOUNDATIONS OF THE GEORGE WASHINGTON BRIDGE

Age	New Jersey Tower		New York Tower		New York Anchorage		New Jersey Anchorage	
	Number of cylinders	Breaking strength, in pounds per square inch	Number of cylinders	Breaking strength, in pounds per square inch	Number of cylinders	Breaking strength, in pounds per square inch	Number of cylinders	Breaking strength, in pounds per square inch
28 days	112	2 200	55	2 990	306	2 480	35	2 960
3 months	32	2 940	14	3 130	93	3 010	15	3 670
6 months	29	2 790	9	4 120	83	3 190	14	4 250
1 year	4	3 230	..	.....	27	3 730	6	4 210
2 years	5	3 760	4	5 320	19	3 420	5	5 210
3 years	3	3 810	1	5 500	17	3 550	4	4 450
4 years	2	4 100	1	5 780	..	.....	..	.....

can be accomplished as the methods of handling concrete from the mixer to its position in the forms are improved so that workability is obtained with a still greater saving in the cement content and with a resulting strength well beyond that demanded by the needs of the structure under construction.

PERSONNEL

The work of the construction of the New Jersey pier and tower foundations, known as Contract HRB-2, was performed by the Silas Mason Company. Francis Donaldson was Chief Engineer and H. L. Meyer, Superintendent for the Company. The excavation of the New Jersey anchorage was done by Foley Brothers, Incorporated, Contract HRB-3, for which, Carl Swenson was General Manager, John Holmes, Superintendent, and G. H. Wilsey, M. Am. Soc. C. E., Chief Engineer.

Both the New York tower foundation and the New York anchorage were combined in one contract, HRB-4, the Arthur McMullen Company, Contractor. For the contractor the work was under the general direction of E. P. Palmer, M. Am. Soc. C. E.; H. B. Gates, Assoc. M. Am. Soc. C. E., was Chief Engineer.

Of the Port Authority Staff, in charge of the work, F. E. Cudworth, M. Am. Soc. C. E., was Resident Engineer, and A. H. Morrill, Assoc. M. Am. Soc. C. E., Assistant Resident Engineer, of the New Jersey section during the period of construction of the New Jersey tower foundations and the excavation of the New Jersey anchorage, and Robert Hoppen, Jr., M. Am. Soc. C. E., was Resident Engineer, and R. F. Wheadon, Assoc. M. Am. Soc. C. E., Assistant Resident Engineer, of the New York section during the construction of the New York tower piers and New York anchorage.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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TRANSACTIONS

Paper No. 1823

GEORGE WASHINGTON BRIDGE: CONSTRUCTION OF THE STEEL SUPERSTRUCTURE

BY E. W. BOWDEN,[1] AND H. R. SEELY,[2]
ASSOCIATE MEMBERS, AM. SOC. C. E.

SYNOPSIS

The erection of the towers, the cables, and the floor steel of the George Washington Bridge is described in this paper. The general characteristics of the steel superstructure are enumerated briefly, after which a detailed treatment is given of the erection methods and of the equipment. Each of the three distinct parts of the work—the towers, the cables, and the floor steel—is discussed separately in the order in which the work was done.

The erection of the large quantities justified unusual attention to detail and refinement in equipment. The entire project proceeded according to a carefully prepared construction program which was substantially adhered to throughout. The equipment used and methods pursued are, therefore, of special interest.

INTRODUCTION

Two contracts for the steelwork for the main structure of the bridge—one for the towers and floor steel combined, and the other for the wire cables, suspenders, and anchorage steelwork—were executed November 4, 1927. In all, fourteen bids were received for the work from five different contracting companies.

Bids had been called for on both wire cable and eye-bar cable designs. Under both designs bidders were permitted to submit proposals for the entire steel structure, or for the towers and floor steel as one contract, and the cables, suspenders, and anchorage steelwork as another. An opportunity was given to base the bids upon alternate programs of erection—"simultaneous" or

[1] Asst. to the Chf. Engr., The Port of New York Authority, New York, N. Y.
[2] Res. Engr., Central Section, Hudson River Bridge, for The Port of New York Authority, Fort Lee, N. J.

"successive." Simultaneous erection contemplated the construction of the four cables complete before work was begun on the erection of the floor system. Successive erection contemplated the construction of one of each pair of cables prior to the erection of the floor steel and the construction of the remaining cables immediately thereafter.

For the wire cable design, alternate arrangements were also offered. Cables could be built with the two cables of each pair arranged side by side or one above the other.

The lowest combination of bids received was that for the towers and floor steel as one contract and the wire cable construction as another; the latter bid was based on cables arranged side by side to be built by the program of successive erection. By mutual agreement, this was changed later to the program of simultaneous erection.

The contract for the towers and floor involved furnishing and erecting approximately 40 200 tons of structural steel in the towers and approximately 19 100 tons in the floor system. The contract for the cables and anchorage steelwork involved furnishing and erecting approximately 28 300 tons of cable wire, 170 200 lin ft of $2\frac{7}{8}$-in. steel wire suspender rope, about 6 000 tons of structural steel and castings, and the furnishing, but not the erection, of about 2 200 tons of structural steel (eye-bars, pins, and girders) for the New York anchorage.

Description

The steel superstructure has been fully described in previous papers of this series dealing with design. However, brief descriptions of the towers, cables, and floor steel are included with this paper, in order that the reader may have clearly in mind the features essential to an understanding of the erection procedure.

The Towers.—Each steel tower[a] is essentially an assembly of four rigidly connected bents. A pair of columns composes each leg of a bent, the inside column being vertical in a plane at right angles to the axis of the bridge, while the outside column slopes in toward the top with a batter of $\frac{3}{8}$ in. per ft. The columns forming a leg are spaced 47 ft 6 in. from center to center at the pedestals and 38 ft 3 in. at the tower top while a constant distance of 106 ft between centers is maintained throughout the height of the inside columns of the two legs. All bents batter toward the transverse axis of the tower, the outside bents, $\frac{3}{8}$ in. per ft and the inside bents $\frac{1}{8}$ in. per ft. The spacing between the bents at the tower base—18 ft 8 in. from center to center—is thus reduced to 12 ft 6 in. at the tower top.

The towers are 559 ft 6 in. high from the top of the pier (15 ft above mean high water) to the top of the tower at the base of the cable saddle grillage. Vertically, the towers are divided into twelve panels which have different dimensions for the two towers because the roadway level at the New Jersey tower is 16 ft 6 in. higher than at the New York tower. The panel heights vary from a maximum of 56 ft 6 in. to a minimum of 38 ft 3 in.

[a] "George Washington Bridge: Design of the Towers," by Leon S. Moisseiff, M. Am. Soc. C. E., see p. 164.

The columns are single-cell box-sections, with webs or extensions projecting at the corners, and are of varying sectional areas. The maximum section is found in the inside line of columns in the top panel and is 1 296 sq in. The sectional area of all columns in the lowest panel is 717 sq in. The columns, are spliced immediately above each panel point. They are of silicon steel, but the bracing members are principally of carbon steel.

The Cables.—The four 36-in. cables are in pairs spaced 106 ft. apart; the cables forming a pair are arranged side by side, 9 ft between centers. Each cable contains 26 474 cold-drawn galvanized steel wires, 0.196 in. in diameter, which are grouped into 61 strands of 434 wires, each strand being independently attached at the anchorage to a pair of eye-bars forming the upper links of a chain extending down into the concrete of the anchorage, to terminate at anchor girders.

At the towers and anchorage buttresses, the cables are carried on cast-steel saddles. Those at the towers are of unusual size, 8½ by 28 ft in plan, and weigh approximately 180 tons. Rollers under the saddles at the towers provide for motion due to changes in loading conditions during construction. Rockers care for the more limited motion of the anchorage saddles due to these changes, combined with temperature changes during and after construction.

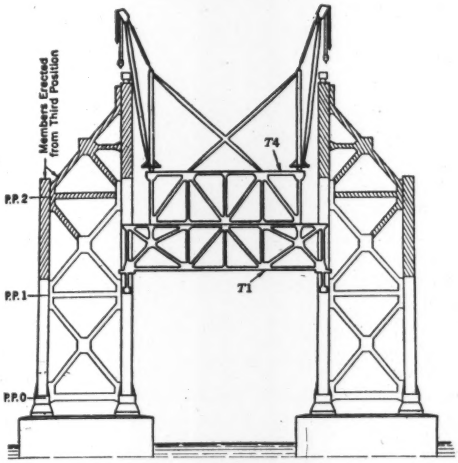
Cast-steel cable bands, bolted together in halves, encircle the cable at each panel point. Two grooves in each band retain the suspender rope loops, thus providing four parts of 2⅞-in. wire suspender rope from each cable for attachment to the floor-beam at a panel point.

The Floor Steel.—As now in place, the floor system is a single deck only, without stiffening trusses. It is 118 ft in over-all width, providing for a roadway 90 ft wide between outside curbs, the roadway space being divided into three parts by intermediate curbs, the two outside parts only being concreted at the present time. On either side of the roadway are sidewalks for pedestrians.

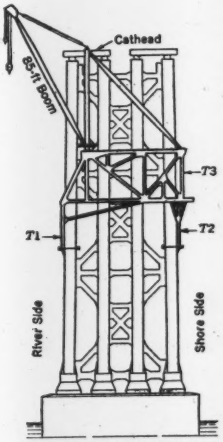
The main transverse floor-beams—plate girders, 10 ft deep and 118 ft long—are spaced at 60-ft intervals and are attached to the socketed ends of the suspender rope groups. Eight longitudinal roadway stringers in each panel, varying in depth from 5 ft 4 in. to 5 ft 8 in., frame into the floor-beams and carry secondary transverse 16-in. I-beams, spaced at 5 ft 2-in. intervals. The main reinforcement of the concrete deck consists of 6-in. bulb-beams, spaced 15 in. from center to center running longitudinally, supported by the secondary transverse beams. A longitudinal beam adjacent to the roadway curb and a fascia stringer support the sidewalk beams, which are spaced at 3 ft 9-in. intervals.

Stress expansion in the roadway and sidewalk is provided at each panel. The longitudinal members framing between the floor-beams are riveted to the floor-beam at one end, but are left free to move at the other.

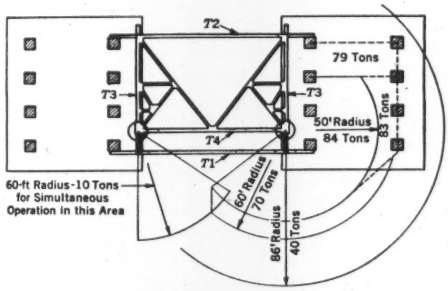
The roadway floor-beams are held rigidly spaced by a continuous wind chord which passes through the floor-beam about 6 ft from each end. The wind chord is designed to become the upper chord member of a light stiffen-



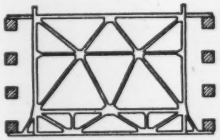
TRAVELER IN THIRD POSITION



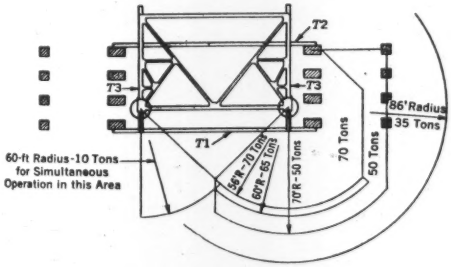
SECTION NEAR CENTER LINE OF TOWER



TRAVELER AT BASE OF TOWER
Derrick Capacities Below Panel Point 6



BRACING AT BOTTOM OF LONGITUDINAL TRUSSES T3



TRAVELER AT TOP OF TOWER
Derrick Capacities Above Panel Point 6

FIG. 1.—DETAILS OF TRAVELER FOR TOWER ERECTION.

ing truss system when the lower deck is added at a future time. The gusset-plates for connection with the other members of the stiffening truss are attached to the chord, but are left blank to be drilled when the stiffening trusses are added.

Expansion joints for temperature changes for the floor system are provided at the towers. The wind truss, which consists of continuous lateral bracing in the plane of the wind chord, terminates at the tower in a strut member on the center line of the bridge. The wind reactions are transmitted from this member to the tower steelwork by guides between which the strut floats freely under normal conditions. The stringers in the panel adjacent to the tower are designed to slide freely on the steel framing of the tower floor. At the roadway level, bar-grating expansion joints are provided.

Tower Erection

Erection Equipment.—The conditions peculiar to the towers, especially the wide spacing between the inside column lines and the comparatively large area occupied by the columns composing each leg, led to the design of an erection traveler with two stiff-leg derricks, one for the erection of each tower leg.

The derricks were carried by a steel framework that spanned the opening between the tower legs, as shown in Fig. 1. Two transverse trusses of this framework, $T1$ and $T2$, carried the reactions to the inside columns on the river and shoreward faces of the towers. These trusses were braced together by adjustable struts and cable guys. A pair of longitudinal trusses, $T3$, parallel with, and 13 ft inside, the center lines of the inside column rows, were framed above the transverse trusses and were carried by them. The masts of the stiff-leg derricks were set up on these longitudinal trusses, approximately 12 ft back from the river face of the tower, at the point of framing a transverse connecting truss, $T4$. The transverse and longitudinal trusses carried the reactions of the stiff-legs of the derricks. Bracing members were provided in the upper and lower chord planes of the longitudinal trusses, $T3$.

The batter of the columns at the tower faces introduced a special problem in the traveler design. The distance between the outside faces decreased a varying amount at each panel, the total decrease from the lowest to the highest positions of the traveler amounting to 15 ft 8 in. It was thus necessary to decrease the spacing between the transverse trusses that carried the reactions to the tower columns, at successive positions of the traveler. To meet this condition rollers were provided at the points of bearing of the lower chords of the longitudinal trusses on the upper chord of the shoreward transverse truss. The arrangement is illustrated in Fig. 2. The shoreward transverse truss, $T2$, was thus movable with reference to the remainder of the system. To facilitate and insure the necessary adjustment two 20-ton telescopic jacks were provided. After each jump of the traveler, before proceeding with steel erection, the longitudinal trusses were lifted from the rollers by means of screw-jacks.

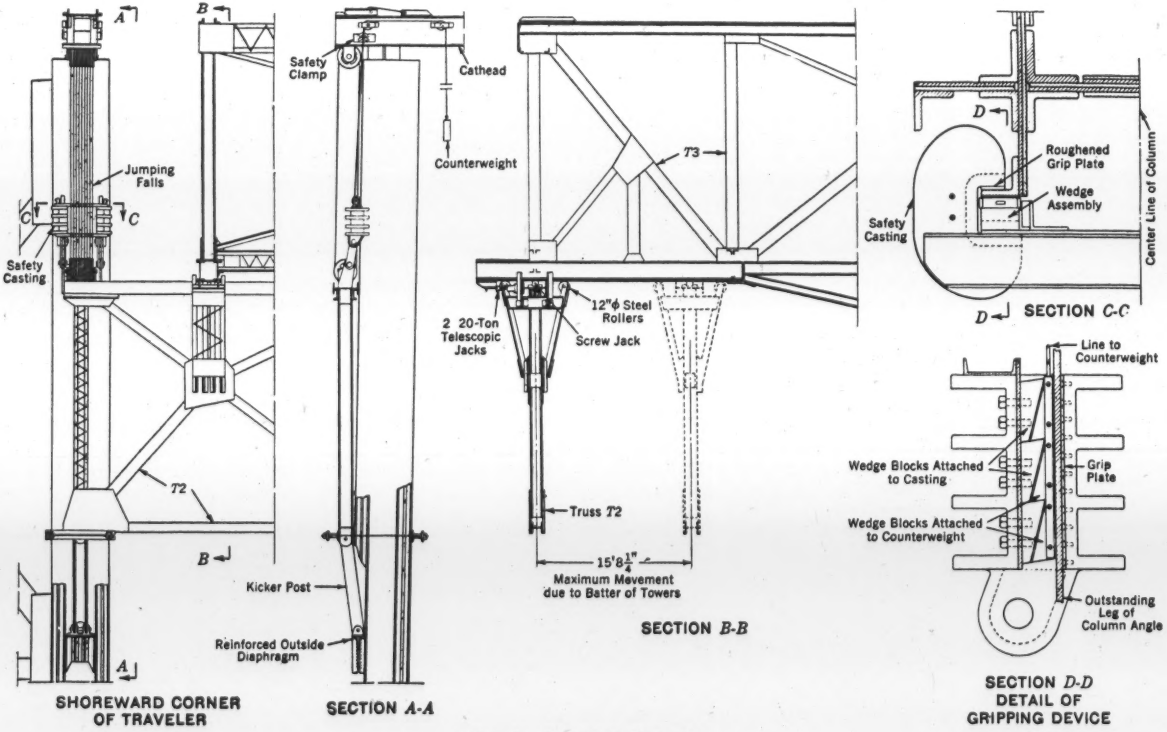


FIG. 2.—TRAVELER DETAILS.

The transverse trusses that spanned between the tower legs were assembled in two stages. As first erected, it was necessary to omit the end posts, a section of the bottom chord, and the end diagonals of each truss, since these members would have fouled the pedestals. Temporary members carried the trusses until the traveler was jumped to a position opposite Panel Point 1. At the New Jersey tower the temporary end members were carried on blocking at the piers. At the New York tower the reactions were carried to rock, near the inside faces of the piers, by means of temporary posts.

The transverse trusses were completed when the traveler was advanced to its first position opposite Panel Point 1, and short kicker posts were added for the purpose of carrying the traveler loads to the tower columns. Throughout their length the columns are stiffened by diaphragms on the inside and between the webs on the outside, at intervals of $6\frac{1}{2}$ to 7 ft. The outside diaphragms, when properly reinforced, were utilized to support the traveler, forming seats for the kicker posts, which were pin-connected to the transverse trusses. An eccentric thrust caused by pulling the post in to engage the shelf angle was overcome by a yoke of channel sections and rods which passed around the column and secured the transverse truss against the column at the pin connection. The total weight of the traveler was approximately 320 tons. The derricks were equipped with 85-ft booms. Their loading capacities are given in Fig. 1.

It was permissible for both booms to handle their maximum loads simultaneously, provided that in so doing neither should encroach upon an area directly in front of the traveler. Within that area the maximum permissible load for either boom was 10 tons if, at the same time, the other boom was handling its maximum load. A boom extension was also provided, good for 15 tons in any position.

Two engine units for the operation of the two derricks were set up on concrete bases at the ground level, about 150 ft shoreward of the tower. Each unit was driven by two 150-hp, 440-volt, three-phase, alternating-current motors, and each unit comprised seven drums—two for the main load falls, two for the boom falls, one for the auxiliary load falls, and two for jumping the traveler. The seven-drum engine was practically a coupling of two engines—one of four drums and one of three drums, each engine with one 150-hp motor.

The boom falls consisted of twenty-two parts of $\frac{7}{8}$-in. cable and the auxiliary load falls of two parts of 1-in. cable. The main load tackle consisted of twelve parts of $\frac{7}{8}$-in. steel cable in a single length of 9 800 ft reeled on two drums so that each end of the line became a lead line capable of a speed of 250 ft per min under a tension of 20 000 lb. When the drums were operated together this had the advantage of double the hoisting speed and better distribution of the hoisting load among the parts. Even with this arrangement, loads hoisted to the top of the tower on the main falls required approximately 17 min for the trip.

Frictional losses were reduced by the use of roller-bearing sheaves for the main load, boom, and auxiliary load falls. Bull-wheels and small electrically operated engines, on the traveler, were used for swinging the booms.

The traveler was "jumped" or advanced from panel to panel by means of four beams of box-section, termed "cat-heads," set up on the tops of the inside column rows. (See Fig. 2.) Each beam was carried by two columns and projected far enough beyond the outside column to support an 18-part 1-in. wire-rope falls above an end of one of the two main transverse trusses. The attached projecting splice material at the tops of the columns made it impossible to rest the beam directly upon the milled ends of the sections. Two sets of plates were framed transversely to the box-section and held it above the projecting column splicing material. The plates were milled and bore upon the milled ends of the column sections. At points where the bearing plates of the "cat-head" were of less thickness than the column web material, shim plates were used to fill the space between column splice-plates; and the bearing plates were then bolted temporarily to the splice-plates of the column.

An interesting feature of the traveler equipment was the safety device provided at each of the four corners of the traveler to prevent it from dropping in case the jumping falls failed during the process of advancing from one position to the next. The device was designed to function by forcing wedge-blocks into contact between the outstanding legs of column angles and steel castings fitting around these angles, the falling motion causing the gripping action. The castings were pin-connected by links to the upper corners of the transverse trusses. Two sets of wedge-blocks worked together; one set was tap-riveted to the casting while the other set was made to follow along during the jumping operation by means of a counterweighted line that passed through sheaves at the "cat-head." The line passed through a safety cable clamp which permitted movement of the counterweighted line in the upward direction only. In case of a reversal of motion, the gripping action of the safety clamp held the line and forced the wedges into contact. Fortunately, the functioning of this safety device was never required.

A satisfactory system of electric signals was devised for the operation of the derricks. A signalman at the point of erection operated a small portable panel of electric push-buttons which controlled signal lights at the engines. For each drum, or set of drums, white and red lights were provided. The engineer hoisted the load or slacked off only while the corresponding indicator bulb remained lighted. This system of signals was especially effective and accident-proof since the maintenance of a light required a continuous pressure on the button in the hands of the signalman and a discontinuance of the signal in effect, whether by purpose or accident, simply stopped all operation.

When designing this equipment the use of a telephone system with head phones for the engineer had been considered, but was decided against in view of the possibility of serious accident that might result through failure of the order to stop the operation in case of incapacity of the signalman or failure of the circuit.

Provisions for Handling Material.—The tower locations are such as to facilitate the delivery of materials by car-float. The piers of the New Jersey

tower are just offshore, with ample depth for mooring car-floats on the riverward side. The steel sections were thus picked directly from the cars and hoisted to position in the tower. Clusters of piles were located north and south of the piers to assist in the manipulation of the car-floats.

The foundation piers of the New York tower are situated on the rocky point of Fort Washington Park, approximately 80 ft back from the water's edge. Here it was necessary to provide docking facilities and a means of handling the materials from the car-floats to a position within reach of the tower traveler. The construction of a substantial temporary unloading dock in front of the tower offered a problem because of the sharply sloping rock that lies bare at that point. A rock-filled crib type of construction was chosen. The cribs were anchored in the rock bank and tied back to the tower foundation. The broken rock-filling was placed to a height 3 ft above high water. When planked over and sheathed on the river face, a very satisfactory mooring wharf was obtained.

A stiff-leg derrick was erected in position to handle the steel from car-floats to traveler derricks. By this means several carloads of material could be unloaded and stored in front of the tower. However, the main point of storage for the materials of both towers and, in fact, the only point of storage for materials for the New Jersey tower, was at the Jersey City, N. J., terminal yards of the railroads, approximately ten miles down the river. Here, the fabricated steel was brought and placed in ground storage to be reloaded on cars and placed on car-floats as needed for erection at the bridge site.

Erection of New Jersey Tower.—The two towers were erected simultaneously, requiring a duplication of equipment and practically a separate field organization, all, however, co-ordinated under one general supervision. In the main, the same method of procedure was followed in the erection of the two towers, minor variations being made to suit the particular conditions encountered. The New Jersey piers were completed several weeks in advance of the New York piers, permitting erection of the steel for the New Jersey tower to be started six weeks earlier. In consideration of the foregoing facts the following description of the erection will outline the procedure at the New Jersey tower, afterward noting the points of variation and the progress on the New York tower.

In the course of the description reference will be required to the columns, bents, and panel points. This reference need cause no confusion because of the extremely simple and practical system of designation used on the work. The bents were identified by numeral and the column rows by letter. From west to east the four bents composing the New Jersey tower were numbered 1, 2, 3, and 4, respectively. The enumeration was continued in the same manner for the New York tower bents, which were 5, 6, 7, and 8. The column rows were lettered from south to north, the column rows, *A* and *B*, comprising the south leg of each tower and the rows, *C* and *D*, the north leg. Panel points were numbered from 0 to 12, with the zero panel point directly above the pedestals. Using this system of designation any point on

either tower could be readily indicated, and in a manner which insured against confusion between the towers. For example, Panel Point 10 of the column at the northeast corner of the New Jersey tower was designated *D*-410 (Column Row *D*; Bent No. 4, and Panel Point 10). The same point of the column at the northeast corner of the New York tower would be *D*-810 (Column Row *D*, Bent No. 8, and Panel Point 10).[4]

Before erecting any tower steel on the New Jersey piers, the traveler was assembled by derrick-boats and stiff-leg derrick, the latter set up on the north pier for the purpose. The contractor began assembling the framework of the traveler on May 29, 1928. The trusses and derricks were erected, the engines installed on the shore near the piers, and the work of reeving was completed, ready for the erection of the tower steel by the latter part of June. The first of the sixteen pedestals were set on June 23, 1928.

The pedestal is a built-up structural member, 14 by 14 ft in plan, and 6 ft 6 in. high. It is anchored to the pier by six 2½-in. anchor-bolts, those for the pedestals of the bent at the river face of the tower embedding 12 ft 4 in. in the concrete masonry, and the remainder of the bolts embedding 5 ft 4 in.

The pedestals for the New Jersey tower were delivered to the site completely assembled. In setting, a uniform bearing was obtained by a carefully prepared system of mortar bedding, as shown in Fig. 3. The concrete of the piers had been finished off to a level approximately 1½ in. lower than the required elevation for the base slabs of the pedestals. Corrugated pipes had also been set at the anchor-bolt locations at the time the concrete was placed.

As a first step, a pair of steel screed rails were set, one at either side of the pedestal location. These were carefully adjusted to grade by level-instrument observations and were fixed in position by quick-setting cement. After placing wooden caps over the corrugated pipes that provided the open holes for the anchor-bolts, a stiff mortar of 1 part sand and 1 part cement, of a consistency approximating putty, was spread over the pedestal area, but was kept back about 8 in. from four accurately adjusted shim-plates, or pads, set to receive the pedestal and to insure exact elevation.

The mortar was reduced to a plane approximately ⅛ in. higher than the final elevation for the base of the pedestal. This leveling process was accomplished by rolling and by striking off with a screed board between the screed rails. The wooden plugs were then withdrawn from the anchor-bolt pipes and the bolts, equipped with pilot nuts, were placed and grouted. Finally, the pedestal was picked and lowered into position and brought to bear upon the four corner pads.

These pads are worthy of a detailed description. A ½-in. steel plate was first set to exact elevation. Upon this was placed an assembly consisting of two bronze plates separated by a steel plate with a gib. A ½-in. steel plate was placed in turn on these. Other plates, of various thicknesses, were furnished, such that refinements of 1/32 in. in the adjustment of this corner-pad

[4] See Fig. 9, p. 182.

assembly was possible. The gib-plate permitted the withdrawal of the entire assembly after the mortar had set.

At first, the mortar was made very stiff, and it was necessary to place other pedestals on top of the one being set, in order to force it down upon the corner pads. Later, a softer mix was used, which permitted easier manipulation. Excellent results were secured by this method of mortar bedding. In one case only was it necessary to pick a pedestal that had been set. An error of $\frac{1}{8}$ in. was detected in its adjustment. When lifted from position it was found that the mortar was entirely free of hard spots, air pockets, or voids. The markings of the planing tool on the slab were clearly visible in the hardened mortar bed. The setting of the pedestals for the New Jersey tower was completed about July 1, 1928.

The traveler then erected all the column sections and bracing of the first panel and also the equipment for advancing the traveler, after which it was "jumped" a half-panel length to a position opposite Panel Point 1 (see Fig. 4) where the transverse trusses of the traveler were completed. The traveler then erected the second panel length of columns forming the two inside rows (the *B* and *C* rows), together with the diagonal members necessary for bracing. The jumping tackle was erected on these inside columns and the traveler was advanced a panel length to a position opposite Panel Point 2.

In this position the traveler erected the second panel length of columns forming the two outside rows (the *A* and *D* rows), the remaining bracing of the second panel and the third panel length of the columns forming the two inside rows (the *B* and *C* rows) with their diagonals.

This procedure was repeated for each successive position of the traveler, which advanced a panel length at a time. At the completion of erection at a given position of the traveler, the inside rows of columns were always one section or panel length in advance of the outside rows.

The arch ring and strut of the lower portal steelwork opposite the third panel were erected by the traveler while it was opposite Panel Point 4. The remainder of the framing for this portal was erected with the traveler at Panel Point 7. To erect the portal steelwork, which was directly below the traveler, the stiff-leg of one of the derricks was disconnected temporarily, to allow the boom to be swung around over the central area. The falls were then lowered through the openings between the members of the traveler frame.

Prior to the erection of the portal steelwork, a pair of horizontal spacing struts were erected temporarily at Panel Point 2, connecting the legs of the tower as shown in Fig. 5. It was intended that these spacing struts should assist in maintaining the proper distance between the separate units. However, they were not sufficiently stiff to permit of much compression. Fortunately, the accuracy of the erected material was such that these spacing struts were not actually required. The struts used in this position were to be incorporated permanently in the structure at Panel Point 3. Similar struts from the top portal steelwork were used in temporary position at Panel Points 6 and 8.

FIG. 3.—SETTING PEDESTAL FOR NEW YORK TOWER

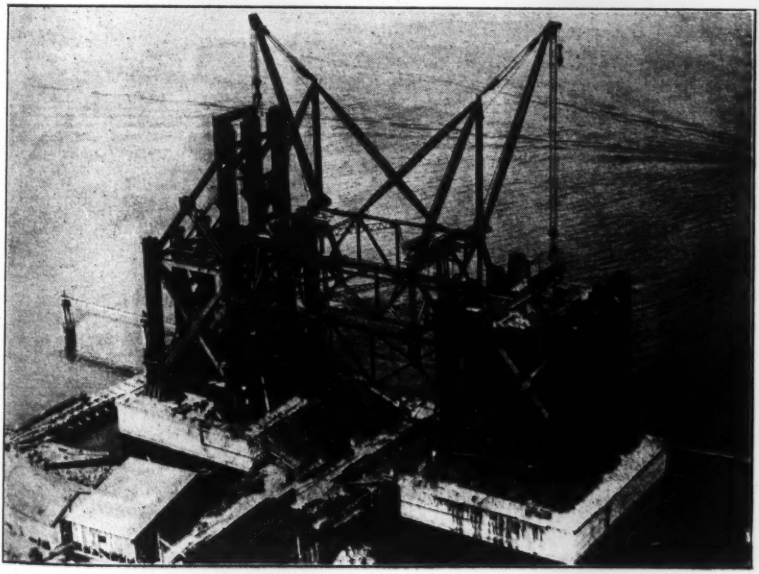


FIG. 4.—NEW JERSEY TOWER TRAVELER OPPOSITE PANEL POINT 1

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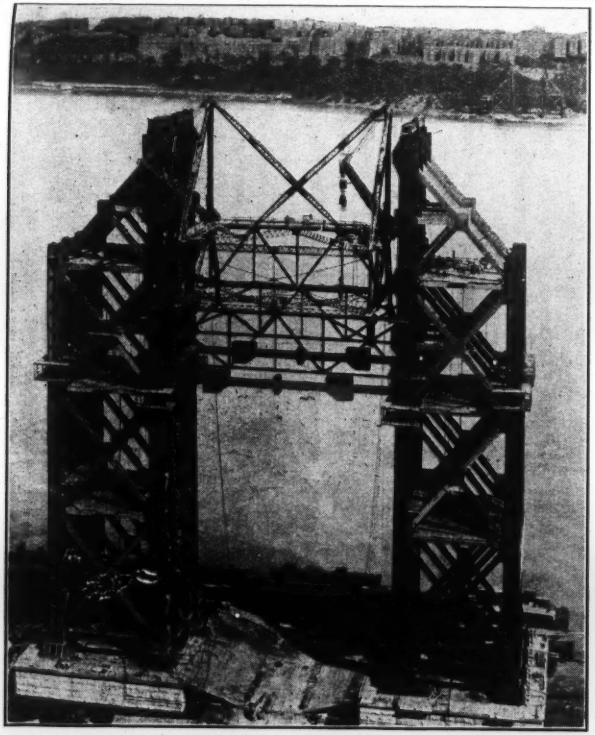

FIG. 5.—ERECTION COMPLETED FOR TRAVELER POSITION OPPOSITE PANEL POINT 3; READY TO ADVANCE TRAVELER TO NEXT POSITION.

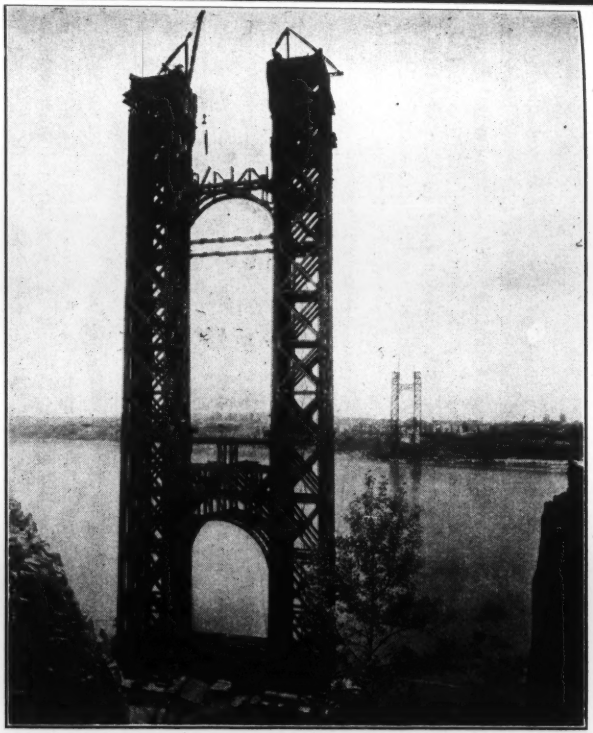

FIG. 6.—ERECTING UPPER PORTAL STEELWORK.

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Column sections were delivered for erection with all gusset-plates for struts and diagonals riveted in place. The splice-plates were riveted to the tops of the column sections although the upper rows of rivets were omitted and the splice-angles on the outstanding legs were left loose. Erection of a column section was thus accomplished by lowering or "knifing" in between the splice-plates attached to the column section previously erected. Care was necessary, in order to insure satisfactory joints since, in accordance with the provisions of the specifications, all contact surfaces had received two coats of red lead paint. The entering web usually scraped paint from the inside of the splice-plates, causing an accumulation of the material on the lower contact surface. When the member was within an inch of contact, the lowering of the column section was stopped long enough to permit cleaning the accumulated material from the contact surfaces, by blowing with compressed air. A careful inspection of contact joints was made in all cases, and riveting was permitted only when the opening at point of contact was less than 0.003 in. In a few cases, riveting was delayed until considerable weight of steelwork had been erected above.

Erection of New York Tower.—The New York tower was erected along lines similar to those used in the erection of the New Jersey tower. The first pedestals were set on August 6, 1928. It will be recalled that the pedestals for the New Jersey tower were delivered to the site completely assembled. The pedestals for the New York tower, however, were delivered in two sections for assembly at the site. These were assembled on heavy girder beams with the sections of the base slab in a true plane, and in this position the sections were fitted and riveted together. The method of bedding and adjusting the pedestals by means of corner pads was the same as that for the New Jersey tower.

The erection equipment, the positions during erection, and the methods of procedure were similar in general to those on the New Jersey side, the principal difference came about by reason of the necessity of handling the materials from car-float to tower base by means of an unloading derrick.

It would seem that this use of an additional piece of equipment, together with the personnel required for its operation (a force of about ten men), would cause an increase in the cost of erection. Such was not the case, however, because the unloading derrick was found to effect an actual saving in the time of erection. For one thing, this derrick was of assistance in picking column sections from a horizontal to a vertical position, an operation which otherwise required the use of the second traveler derrick.

The erection of the lower portal steelwork was effected in a slightly different manner from that used in New Jersey, where, it will be recalled, the boom was swung between the stiff-legs so that the falls could be lowered through the truss work of the traveler. In New York, special tackle was hung from the lower part of the traveler frame and used to hoist and drift the various members into position. The latter method appeared to be the more economical in view of the fact that the time required in placing the falls in correct position through the framework more than offset the advantage this method of portal erection seemed to offer.

Observations at Panel Point 10.—The Port Authority specifications provided that the top sections of the columns should be finished in the shop after the erection of the remainder. This permitted the determination of the proper lengths. Because of the deep girders framing into the column sections in the top panel, complications would be found in an attempt to compensate for any discrepancies in column length in this panel. It was decided, therefore, to make such adjustments in the second panel from the top, and the contractor prepared his schedule with the view of reaching Panel Point 10 before the arrival of severe winter weather. At this point the schedule called for a discontinuation of erection operations for the duration of the winter, permitting time for taking measurements to determine the actual relative elevations of the tops of the column sections, and giving an opportunity for finishing the column sections, 10-11, to compensate for the variations found.

In the New Jersey tower, 15 782 tons of structural steel were erected to Panel Point 10 between June 27, and November 1, 1928, a period of approximately eighteen weeks, while, in New York, 15 785 tons were erected between August 6 and November 24, 1928, a period of fifteen weeks.

The observations at Panel Point 10 were made by level instrument, rod, and target. Knowledge of the exact height of the steelwork above the established datum plane was of no practical value, and no attempt was made to determine such elevation. The observations sought only to establish the variations which had accumulated between the columns composing a tower leg.

The readings were somewhat difficult to make because of the attached splice material and because each of the milled surfaces sloped in at least one direction and one-half of them in two directions, due to the column batter. They were taken on the milled surface of each column at four points—two on the transverse and two on the longitudinal center line of the column. When averaged, these readings gave the elevation at the column axis. They were taken separately by two parties on each tower, in order to eliminate personal error, and were made under conditions that would insure, as far as possible, a uniform temperature in all columns; that is, in the early morning or under cloudy conditions. The variations are given in Table 1.

In consideration of the fact that the relative differences in Table 1 were accumulated in the erection of approximately 500 ft of column height, the the small amount of discrepancy found between the columns attests to the high degree of accuracy of the shop work, and the pains taken by the workmen and inspectors in erecting this steel.

Adjustments were made at the shop in the lengths of the columns between Panel Points 10 and 11. The lowest column at Panel Point 10 was assumed to be at plan elevation and to require no correction. All other columns were made shorter by the amount of over-run as indicated.

The usual procedure in preparing the splicing material in the shop was to assemble any given column section with the section to be erected next above it, and to ream the splice material assembled to the columns.

TABLE 1.—VARIATIONS IN COLUMN HEIGHTS AT PANEL POINT 10
(Readings are in 32nds in. above the lowest column)

Tower	Column	Variation	Column	Variation	Column	Variation	Column	Variation
New Jersey	*A1*	+4	*A2*	+6	*A3*	+6	*A4*	+6
	B1	+1	*B2*	+3	*B3*	+4	*B4*	+3
	C1	+4	*C2*	+3	*C3*	+1	*C4*	+4
	D1	0	*D2*	0	*D3*	Lowest	*D4*	+2
New York	*A5*	+5	*A6*	+5	*A7*	+4	*A8*	+2
	B5	+7	*B6*	+5	*B7*	+4	*B8*	+5
	C5	+5	*C6*	+5	*C7*	+5	*C8*	+5
	D5	0	*D6*	Lowest	*D7*	+4	*D8*	0

The holes for connecting the splice material to the lower section (9-10) of the columns at Panel Point 10, were handled in the usual way; that is, reamed with splice material assembled to the columns. The holes in the upper half of the splice material were reamed full size (1 1/8 in. in diameter), to a metal template. The splice-plates and angles were then riveted or bolted for shipment. Later, when the columns in Panel 10-11 had been milled to proper length, the holes for the splice at the bottom of the columns were drilled and reamed full size to the same metal templates. It was intended that the holes should be reamed over-size for larger rivets if found necessary after erection, but such action was unnecessary.

On account of changing the lengths of the columns, some adjustments were required in the connecting bracing members. This was taken care of by varying the position of the groups of open holes in the gusset-plates.

Erection Above Panel Point 10.—Erection was resumed on the New York tower on March 26, 1929, and on the New Jersey tower on April 8, 1929. With the exception of the portal steelwork, the erection of the New York tower was completed to Panel Point 12 during the week of May 7; the last two panels were in place on the New Jersey tower on May 29.

The traveler was then dismantled at Panel Point 12. The procedure for dismantling the traveler and erecting the upper portal steelwork was practically identical for the two towers. A stiff-leg derrick was erected by the traveler on one leg of the tower, and one of the pair of traveler stiff-leg derricks was moved to the other tower leg. These two derricks then dismantled the traveler and erected the members of the upper portal, completing it during the latter part of June. (See Fig. 6.)

The contractor for the towers, as sub-contractor for the cable contract, erected the grillage steel, footbridge ropes, cable saddles, and cable-spinning equipment on the tower tops. Accordingly, when the portal was completed, the second stiff-leg derrick of the traveler, which had been lowered to the ground, was rigged for use at the top of the tower. Sills were added, and its boom was increased 20 ft, to a length of 105 ft. This derrick was set in position on the portal bracing near the center line of the tower and was used to dismantle the derricks on the tower legs. Its capacity was 85 tons at a radius of 60 ft.

Fitting and Riveting.—No difficulty was experienced in the field in erecting and fitting the steelwork because the shop procedure was so carefully planned and executed. The special conditions at Panel Point 10, which made impossible the shop assembly of adjacent column sections, have been considered. Similarly, special conditions at Panel Point 6 made impossible the assembly of the sections in bearing at that point.

Because of the large quantity of steel to be fabricated in the limited time, the part of each tower below Panel Point 6 was fabricated in the shops of a sub-contracting company. The splice material at Panel Point 6 was handled in a manner similar to that described at Panel Point 10, except that the holes for connection to the column sections above Panel Point 6 were drilled to $\frac{15}{16}$ in. and reamed to $1\frac{1}{16}$ in. after erection.

During the course of erection of both towers, instrument observations were made at frequent intervals to check the verticality. Considering the limits of probable error of observation, the alignment was satisfactory in all cases. The accuracy of the shop work had been such that at no time was it necessary to resort to any drifting or "guying" to change the alignment.

Riveting was begun on the New Jersey tower when the first two panels of steelwork had been erected, and on the New York tower, when the first three panels had been erected. Thereafter, the riveting on both towers followed closely behind the erection. Shortly after the discontinuation of erection for the winter months, riveting was likewise stopped. Riveting on the New Jersey tower was discontinued on November 28, 1928, to be resumed on April 8, 1929. On the New York tower the riveting was discontinued on December 8, 1928, to be resumed on April 8, 1929. With the exception of a few scattered points left open because of cable-erection equipment, the riveting was completed on both towers during August.

The rivets were of carbon steel, 1 in. in diameter for the main members, although a few $\frac{7}{8}$-in. rivets were used in the portals; a total of 488 000 field rivets was driven in the New Jersey tower and 498 000 in the New York tower. They were heated by means of soft coal forges. Other methods of heating were tried, such as by electric-heating units and oil forges, but were found to be less satisfactory than the ordinary forges. All riveting was done by pneumatic hammer, and "holding on" was likewise done by a pneumatic tool wherever possible. An air compressor, driven by a 125-hp electric motor and having a piston displacement of 675 cu ft per min of free air, was located near the base of each tower. A pressure of about 100 lb was supplied at the tool.

An average of nine or ten gangs of riveters of four men each were engaged on the work on each tower, although as many as thirteen gangs were employed at one time. The men worked $5\frac{1}{2}$ days per week, barring unfavorable working conditions, and averaged more than 300 rivets per gang per day. Under favorable conditions much higher rivet counts were made. The average number of rivets rejected was 2.7% in New Jersey and 2.0% in New York for the entire period of the riveting.

Special precautions were taken to protect the men engaged in the work of fitting and riveting. Planks, 4 in. in thickness, were used to construct a floor over the entire area of the tower leg at each panel point immediately after erection. This floor remained in place until all rivets in the panel were driven. Scaffolds with timber floor and roof on steel frames were hung on the outside column faces. These were protected with hand-railings and were equipped with ratchet winches so that they could be drawn up a panel length at a time by hand operation. Access to the point of steel erection was usually by means of a large cage hoisted from the tower base by one of the traveler derricks. Access to the various points on the tower during riveting operations was facilitated by an elevator installed in one tower leg. The guides and hoist for the elevator were advanced from time to time as erection progressed. However, the highest point reached by the elevator was several panels below the top of the steelwork and ladders were used to cover the additional distance.

Erection of Cables and Suspenders

Equipment for Unloading and Handling Materials.—The delivery of materials not only for the permanent structure, but for the temporary facilities necessary for the erection of the permanent structure, presented quite an unusual problem. The greater part of all the material required eventual delivery to the anchorages several hundred feet back from the waterfront and approximately 200 ft above the river level. Railroad connections to the site, particularly on the New Jersey side, were not to be found, and topographical features on both sides of the river made it rather impractical to consider trucking from the nearest railroad terminals. The contractor, therefore, adopted the only logical method—delivery of materials by water, on barges and car-floats, to or adjacent to the bridge towers, supplemented by cableway transportation to the anchorages.

On the New Jersey side provisions for mooring barges and car-floats were in place as installed by the contractor for the towers and floor steel. Two Chicago booms, 125 ft. in length, attached to the north and south faces of the tower, were used for unloading all material. They were operated by electrically driven hoisting engines placed on the shore with manila rope block-and-fall swing lines, and were capable of placing material on a temporary dock constructed north of the north pier and on a cantilever platform erected on the south side of the tower at Panel Point 0. The south boom was also long enough to reach a material track constructed along the shore line. A light, hand-operated derrick car on this track made it possible to use a level area of ground south of the bridge for storage purposes. This area was particularly necessary for the storage of the footbridge deck.

From the base of the tower, materials were hauled up the face of the Palisades, by means of a cableway with a capacity of 15 tons. A $2\frac{3}{4}$-in. track rope was attached to Tower Column *D*1, at the northwest corner, just below Panel Point 1, and passed over a 65-ft structural steel tower erected on top of the Palisades north of the approach cut. The track rope was anchored with a $1\frac{1}{8}$-in. wire line block and falls by means of which the sag of the

rope could be adjusted as desired. A material track was then used to transport the material from the cableway to a 10-ton crane operating on a runway erected across the approach cut over the west half of the anchorage floor steel. At the east end of the floor steel, over the areas not reached by the crane, materials were handled by a guy derrick.

On the New York side, the contractor elected to make use of a dock, south of the tower, constructed by the contractor for the anchorage. A stiff-leg derrick was erected at the north end of this dock and was capable of unloading all material, placing it either on a loading platform south of the south tower pier, or on a material track extending past the east face of the south tower pier. A caterpillar crane was used for distributing materials for storage on areas not reached by the stiff-leg derrick. As was the case on the New Jersey side of the river, a cableway was used to transport materials from the track at the tower to a second track on the ground between the buttresses of the anchorage. One end of the cableway was attached to Tower Column *A*8, at the southeast corner, just below Panel Point 2, and the other end was attached by a block-and-fall to a special structural steel anchor embedded in the concrete of the anchorage block. A stiff-leg derrick erected on top of the anchorage block on the center line of the bridge served to hoist the material from the material track and distribute it where necessary on the anchorage.

Concurrently with the installation of these facilities, the contractor for the towers proceeded with the erection of the grillage girders and frames for attaching the footbridge ropes, after which he hoisted the footbridge ropes and erected the cable saddles and the steel construction towers.

Grillage Girders.—Each grillage is made up of four longitudinal girders 31 ft long and 5 ft $1\frac{3}{4}$ in. deep, resting directly on top of the lateral plates forming the top plane of the tower bracing. A 6-in. slab carries the load of the saddle from the rollers to the longitudinal girders. As delivered in the field for erection, the entire grillage for each saddle was assembled in two sections which, when erected, were joined on the transverse center line of the tower. The girders of the sections were joined by a butt-stiffener splice. The two sections of slab had no splice.

Little difficulty was encountered in setting the grillages so that the slab would come in a true plane. In one case, because of a slight warp in the grillage section, it was necessary to omit the riveting of one butt-splice temporarily until after the load of the cable had remedied the condition.

Footbridge Ropes.—The construction of parallel wire bridge cables, with the possible exception of those for very short spans, always requires the use of working scaffolds or footbridges. For economy, it has become the practice of contractors to make use of the suspender ropes temporarily, to support these footbridges. The ropes are thus ordinarily manufactured in sufficient lengths to reach from anchorage to anchorage. After the main cables have been spun, these ropes are then cut into the proper lengths to form the suspenders. In principle, this was the procedure on the George Washington Bridge. However, due to the long span, it was practically impossible to

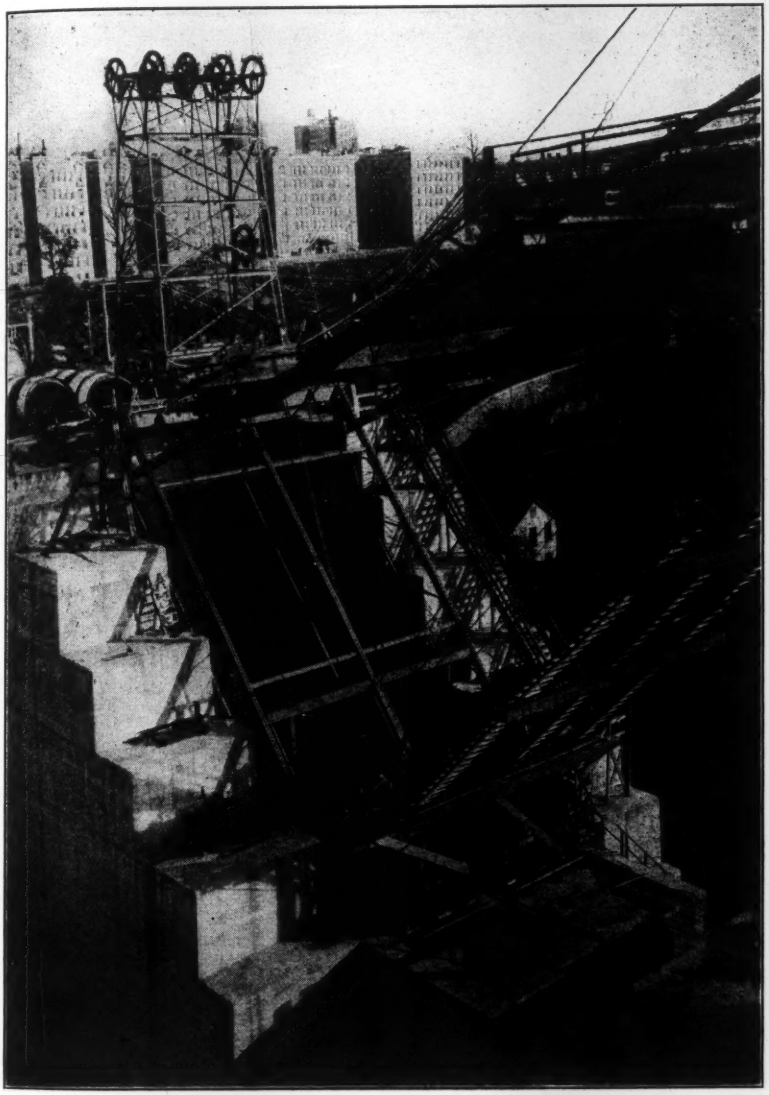

FIG. 7.—SPINNING EQUIPMENT, NEW YORK ANCHORAGE.

Observations at Panel Point 10.—The Port Authority specifications provided that the top sections of the columns should be finished in the shop after the erection of the remainder. This permitted the determination of the proper lengths. Because of the deep girders framing into the column sections in the top panel, complications would be found in an attempt to compensate for any discrepancies in column length in this panel. It was decided, therefore, to make such adjustments in the second panel from the top, and the contractor prepared his schedule with the view of reaching Panel Point 10 before the arrival of severe winter weather. At this point the schedule called for a discontinuation of erection operations for the duration of the winter, permitting time for taking measurements to determine the actual relative elevations of the tops of the column sections, and giving an opportunity for finishing the column sections, 10-11, to compensate for the variations found.

In the New Jersey tower, 15 782 tons of structural steel were erected to Panel Point 10 between June 27, and November 1, 1928, a period of approximately eighteen weeks, while, in New York, 15 785 tons were erected between August 6 and November 24, 1928, a period of fifteen weeks.

The observations at Panel Point 10 were made by level instrument, rod, and target. Knowledge of the exact height of the steelwork above the established datum plane was of no practical value, and no attempt was made to determine such elevation. The observations sought only to establish the variations which had accumulated between the columns composing a tower leg.

The readings were somewhat difficult to make because of the attached splice material and because each of the milled surfaces sloped in at least one direction and one-half of them in two directions, due to the column batter. They were taken on the milled surface of each column at four points—two on the transverse and two on the longitudinal center line of the column. When averaged, these readings gave the elevation at the column axis. They were taken separately by two parties on each tower, in order to eliminate personal error, and were made under conditions that would insure, as far as possible, a uniform temperature in all columns; that is, in the early morning or under cloudy conditions. The variations are given in Table 1.

In consideration of the fact that the relative differences in Table 1 were accumulated in the erection of approximately 500 ft of column height, the the small amount of discrepancy found between the columns attests to the high degree of accuracy of the shop work, and the pains taken by the workmen and inspectors in erecting this steel.

Adjustments were made at the shop in the lengths of the columns between Panel Points 10 and 11. The lowest column at Panel Point 10 was assumed to be at plan elevation and to require no correction. All other columns were made shorter by the amount of over-run as indicated.

The usual procedure in preparing the splicing material in the shop was to assemble any given column section with the section to be erected next above it, and to ream the splice material assembled to the columns.

TABLE 1.—VARIATIONS IN COLUMN HEIGHTS AT PANEL POINT 10

(Readings are in 32 in. above the lowest column)

Tower	Column	Variation	Column	Variation	Column	Variation	Column	Variation
New Jersey	*A1*	+4	*A2*	+6	*A3*	+6	*A4*	+6
	B1	+1	*B2*	+3	*B3*	+4	*B4*	+3
	C1	+4	*C2*	+3	*C3*	+1	*C4*	+4
	D1	0	*D2*	0	*D3*	Lowest	*D4*	+2
New York	*A5*	+5	*A6*	+5	*A7*	+4	*A8*	+2
	B5	+7	*B6*	+5	*B7*	+4	*B8*	+5
	C5	+5	*C6*	+5	*C7*	+5	*C8*	+5
	D5	0	*D6*	Lowest	*D7*	+4	*D8*	0

The holes for connecting the splice material to the lower section (9-10) of the columns at Panel Point 10, were handled in the usual way; that is, reamed with splice material assembled to the columns. The holes in the upper half of the splice material were reamed full size ($1\frac{1}{8}$ in. in diameter), to a metal template. The splice-plates and angles were then riveted or bolted for shipment. Later, when the columns in Panel 10-11 had been milled to proper length, the holes for the splice at the bottom of the columns were drilled and reamed full size to the same metal templates. It was intended that the holes should be reamed over-size for larger rivets if found necessary after erection, but such action was unnecessary.

On account of changing the lengths of the columns, some adjustments were required in the connecting bracing members. This was taken care of by varying the position of the groups of open holes in the gusset-plates.

Erection Above Panel Point 10.—Erection was resumed on the New York tower on March 26, 1929, and on the New Jersey tower on April 8, 1929. With the exception of the portal steelwork, the erection of the New York tower was completed to Panel Point 12 during the week of May 7; the last two panels were in place on the New Jersey tower on May 29.

The traveler was then dismantled at Panel Point 12. The procedure for dismantling the traveler and erecting the upper portal steelwork was practically identical for the two towers. A stiff-leg derrick was erected by the traveler on one leg of the tower, and one of the pair of traveler stiff-leg derricks was moved to the other tower leg. These two derricks then dismantled the traveler and erected the members of the upper portal, completing it during the latter part of June. (See Fig. 6.)

The contractor for the towers, as sub-contractor for the cable contract, erected the grillage steel, footbridge ropes, cable saddles, and cable-spinning equipment on the tower tops. Accordingly, when the portal was completed, the second stiff-leg derrick of the traveler, which had been lowered to the ground, was rigged for use at the top of the tower. Sills were added, and its boom was increased 20 ft, to a length of 105 ft. This derrick was set in position on the portal bracing near the center line of the tower and was used to dismantle the derricks on the tower legs. Its capacity was 85 tons at a radius of 60 ft.

Fitting and Riveting.—No difficulty was experienced in the field in erecting and fitting the steelwork because the shop procedure was so carefully planned and executed. The special conditions at Panel Point 10, which made impossible the shop assembly of adjacent column sections, have been considered. Similarly, special conditions at Panel Point 6 made impossible the assembly of the sections in bearing at that point.

Because of the large quantity of steel to be fabricated in the limited time, the part of each tower below Panel Point 6 was fabricated in the shops of a sub-contracting company. The splice material at Panel Point 6 was handled in a manner similar to that described at Panel Point 10, except that the holes for connection to the column sections above Panel Point 6 were drilled to $\frac{15}{16}$ in. and reamed to $1\frac{1}{16}$ in. after erection.

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Riveting was begun on the New Jersey tower when the first two panels of steelwork had been erected, and on the New York tower, when the first three panels had been erected. Thereafter, the riveting on both towers followed closely behind the erection. Shortly after the discontinuation of erection for the winter months, riveting was likewise stopped. Riveting on the New Jersey tower was discontinued on November 28, 1928, to be resumed on April 8, 1929. On the New York tower the riveting was discontinued on December 8, 1928, to be resumed on April 8, 1929. With the exception of a few scattered points left open because of cable-erection equipment, the riveting was completed on both towers during August.

The rivets were of carbon steel, 1 in. in diameter for the main members, although a few $\frac{7}{8}$-in. rivets were used in the portals; a total of 488 000 field rivets was driven in the New Jersey tower and 498 000 in the New York tower. They were heated by means of soft coal forges. Other methods of heating were tried, such as by electric-heating units and oil forges, but were found to be less satisfactory than the ordinary forges. All riveting was done by pneumatic hammer, and "holding on" was likewise done by a pneumatic tool wherever possible. An air compressor, driven by a 125-hp electric motor and having a piston displacement of 675 cu ft per min of free air, was located near the base of each tower. A pressure of about 100 lb was supplied at the tool.

An average of nine or ten gangs of riveters of four men each were engaged on the work on each tower, although as many as thirteen gangs were employed at one time. The men worked $5\frac{1}{2}$ days per week, barring unfavorable working conditions, and averaged more than 300 rivets per gang per day. Under favorable conditions much higher rivet counts were made. The average number of rivets rejected was 2.7% in New Jersey and 2.0% in New York for the entire period of the riveting.

Special precautions were taken to protect the men engaged in the work of fitting and riveting. Planks, 4 in. in thickness, were used to construct a floor over the entire area of the tower leg at each panel point immediately after erection. This floor remained in place until all rivets in the panel were driven. Scaffolds with timber floor and roof on steel frames were hung on the outside column faces. These were protected with hand-railings and were equipped with ratchet winches so that they could be drawn up a panel length at a time by hand operation. Access to the point of steel erection was usually by means of a large cage hoisted from the tower base by one of the traveler derricks. Access to the various points on the tower during riveting operations was facilitated by an elevator installed in one tower leg. The guides and hoist for the elevator were advanced from time to time as erection progressed. However, the highest point reached by the elevator was several panels below the top of the steelwork and ladders were used to cover the additional distance.

Erection of Cables and Suspenders

Equipment for Unloading and Handling Materials.—The delivery of materials not only for the permanent structure, but for the temporary facilities necessary for the erection of the permanent structure, presented quite an unusual problem. The greater part of all the material required eventual delivery to the anchorages several hundred feet back from the waterfront and approximately 200 ft above the river level. Railroad connections to the site, particularly on the New Jersey side, were not to be found, and topographical features on both sides of the river made it rather impractical to consider trucking from the nearest railroad terminals. The contractor, therefore, adopted the only logical method—delivery of materials by water, on barges and car-floats, to or adjacent to the bridge towers, supplemented by cableway transportation to the anchorages.

On the New Jersey side provisions for mooring barges and car-floats were in place as installed by the contractor for the towers and floor steel. Two Chicago booms, 125 ft. in length, attached to the north and south faces of the tower, were used for unloading all material. They were operated by electrically driven hoisting engines placed on the shore with manila rope block-and-fall swing lines, and were capable of placing material on a temporary dock constructed north of the north pier and on a cantilever platform erected on the south side of the tower at Panel Point 0. The south boom was also long enough to reach a material track constructed along the shore line. A light, hand-operated derrick car on this track made it possible to use a level area of ground south of the bridge for storage purposes. This area was particularly necessary for the storage of the footbridge deck.

From the base of the tower, materials were hauled up the face of the Palisades, by means of a cableway with a capacity of 15 tons. A $2\frac{3}{4}$-in. track rope was attached to Tower Column *D*1, at the northwest corner, just below Panel Point 1, and passed over a 65-ft structural steel tower erected on top of the Palisades north of the approach cut. The track rope was anchored with a $1\frac{1}{8}$-in. wire line block and falls by means of which the sag of the

rope could be adjusted as desired. A material track was then used to transport the material from the cableway to a 10-ton crane operating on a runway erected across the approach cut over the west half of the anchorage floor steel. At the east end of the floor steel, over the areas not reached by the crane, materials were handled by a guy derrick.

On the New York side, the contractor elected to make use of a dock, south of the tower, constructed by the contractor for the anchorage. A stiff-leg derrick was erected at the north end of this dock and was capable of unloading all material, placing it either on a loading platform south of the south tower pier, or on a material track extending past the east face of the south tower pier. A caterpillar crane was used for distributing materials for storage on areas not reached by the stiff-leg derrick. As was the case on the New Jersey side of the river, a cableway was used to transport materials from the track at the tower to a second track on the ground between the buttresses of the anchorage. One end of the cableway was attached to Tower Column *A*8, at the southeast corner, just below Panel Point 2, and the other end was attached by a block-and-fall to a special structural steel anchor embedded in the concrete of the anchorage block. A stiff-leg derrick erected on top of the anchorage block on the center line of the bridge served to hoist the material from the material track and distribute it where necessary on the anchorage.

Concurrently with the installation of these facilities, the contractor for the towers proceeded with the erection of the grillage girders and frames for attaching the footbridge ropes, after which he hoisted the footbridge ropes and erected the cable saddles and the steel construction towers.

Grillage Girders.—Each grillage is made up of four longitudinal girders 31 ft long and 5 ft $1\frac{3}{4}$ in. deep, resting directly on top of the lateral plates forming the top plane of the tower bracing. A 6-in. slab carries the load of the saddle from the rollers to the longitudinal girders. As delivered in the field for erection, the entire grillage for each saddle was assembled in two sections which, when erected, were joined on the transverse center line of the tower. The girders of the sections were joined by a butt-stiffener splice. The two sections of slab had no splice.

Little difficulty was encountered in setting the grillages so that the slab would come in a true plane. In one case, because of a slight warp in the grillage section, it was necessary to omit the riveting of one butt-splice temporarily until after the load of the cable had remedied the condition.

Footbridge Ropes.—The construction of parallel wire bridge cables, with the possible exception of those for very short spans, always requires the use of working scaffolds or footbridges. For economy, it has become the practice of contractors to make use of the suspender ropes temporarily, to support these footbridges. The ropes are thus ordinarily manufactured in sufficient lengths to reach from anchorage to anchorage. After the main cables have been spun, these ropes are then cut into the proper lengths to form the suspenders. In principle, this was the procedure on the George Washington Bridge. However, due to the long span, it was practically impossible to

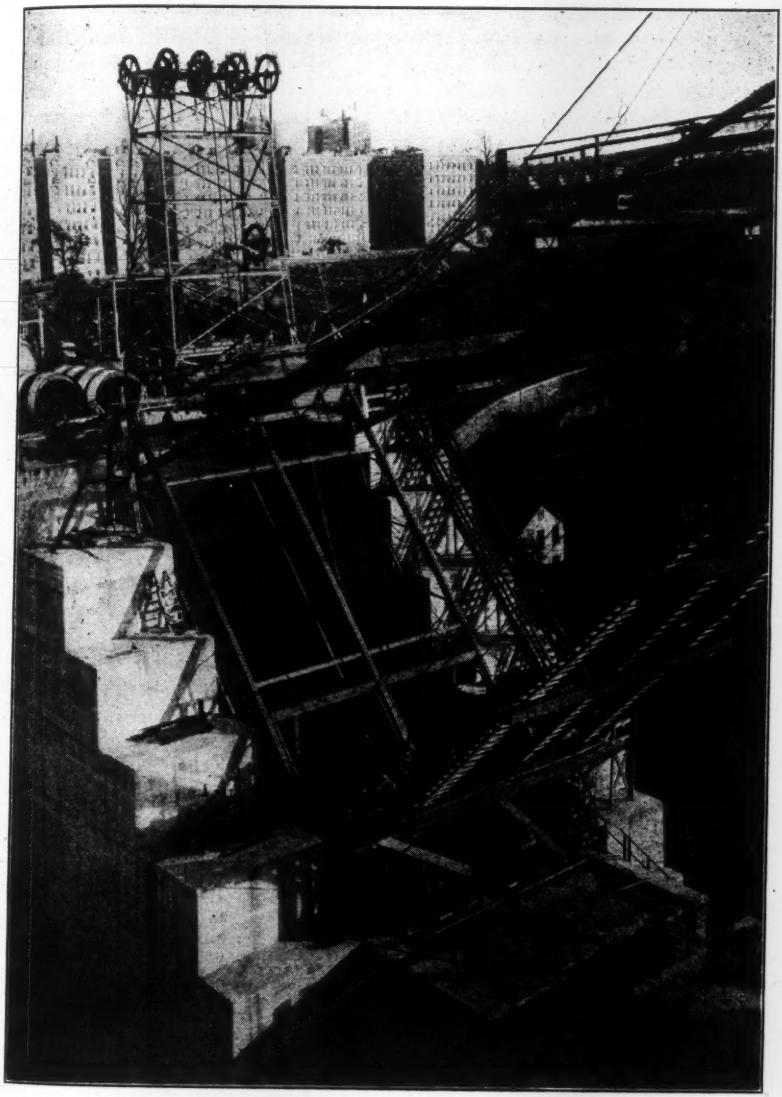

FIG. 7.—SPINNING EQUIPMENT, NEW YORK ANCHORAGE.

handle the rope in one piece of sufficient length to reach from anchorage to anchorage. They were cut in three lengths, one for each side span and the third for the main span. Two groups of nine ropes, $2\frac{1}{8}$ in. in diameter, spaced 22 ft apart, were used to support each footbridge. These groups were arranged with seven ropes in a central hexagonal cluster with one on either side.

The contractor elected to cut the ropes to length and socket them at the shop, and to make the necessary field adjustments for correct sag by varying the relation of the ends of the ropes to their attachments on the towers. Inasmuch as the space available for making these adjustments was relatively small, it was essential that the ropes be cut as accurately as possible to the correct lengths. To secure the required degree of precision, it was necessary to remove all slack inherent in the manufacture of wire rope, to accomplish which each length of rope was pre-stressed under a pull of 200 000 lb at the manufacturing plant, for a period of at least 12 hours. After being measured the rope was cut into the desired lengths, socketed with bail sockets, and placed on reels for shipment.

At the bridge site, the reels of side-span ropes were mounted at the tower bases and the ends were pulled up the hillside to the anchorage where they were attached, by means of structural and cast-steel links, to anchors embedded in the concrete above and on either side of the main cable eye-bars. (See Fig. 7.)

The reels containing the center-span ropes were mounted four at a time on a barge, and after the exposed ends had been connected by pennant lines to their corresponding side-span ropes at the base of the New York tower, the barge was towed as directly as possible to the New Jersey side, the ropes being allowed to unreel and sink to the bottom of the river as shown in Fig. 8.

At the New Jersey tower base the main-span and side-span ropes were connected by pennant lines. The ropes were then in position for hoisting to the tops of the towers, an operation that was accomplished one rope at a time. A short box-girder projecting over the side of the tower at the top, was used to hoist the rope to a point just below the tower top. At this point (the limit of hoisting with the "cat-head" equipment), the load was transferred to the derrick on the top of the tower, which hoisted the ropes the remaining distance and placed them in position in the structural steel frames riveted to the towers on either side of the main cable saddles. During the operation of hoisting the ropes, and while laying them on the river bed, it was necessary to stop all river traffic.

With all the thirty-six ropes in position, the individual lengths in each span were adjusted to hang at predetermined sags, such that with the addition of the footbridge decking and other miscellaneous material, the footbridges would be parallel to the unloaded position of the main cables and about 4 ft below them.

Of necessity, all adjustments were made at the tops of the towers, the attachments of the ropes at the anchorage being such as to preclude the pos-

sibility of adjustments at that point. The side-span ropes were adjusted first, by blocking the sockets of the main-span section and pulling in or slacking off the side-span rope as desired. Fig. 9 shows the hydraulic pulling jacks inserted between the socketed ends of the main and side-span sections of rope to be adjusted.

The desired sags were calculated in advance. Charts were prepared giving the full range of sag corresponding to all changes in span length resulting from tower deflection, and for all reasonable variations in temperature. The chart for the side spans, in addition, gave for each sag the station and the elevation at which a transit should be set up to establish a line of sight parallel to the chord and tangent to the curve having the desired sag.

Plumb-bobs could not be used to measure the deflections in the towers as is customary, because not one of the tower columns was vertical in its normal position and, therefore, could not be used as a well in which to suspend a plumb-bob. The method adopted required the use of transits set up on the piers, sighting on horizontal targets attached to the tops of the towers. The transits for the purpose were equipped with right-angle eyepieces, permitting true zenith observations. These observations were always made in pairs with the horizontal axis of the transit parallel to the center line of the bridge. The instrument was rotated 180° between the first and second of each pair of observations to eliminate instrumental errors. The targets, constructed at the bridge site, were graduated with a pointed stadia design and were artificially illuminated to permit direct readings to within 0.05 ft. They were set with a known relation to points established on the tops of the towers prior to hoisting the footbridge ropes. The plumb or normal positions of the towers had been determined previously by the use of the same transits, observations being made during the hours between midnight and sunrise when the steelwork was most nearly at uniform temperature.

After the side-span ropes had been completely adjusted, the sockets for these ropes were blocked and the main-span ropes were similarly pulled in or slacked off by jacking on one or both towers, as was necessary to group the ropes at the correct sag. The first rope in each group was set tangent to the line of sight of a level instrument set up in one of the towers at the elevation determined from the chart for the observed temperature and tower deflections. A target was also set at the same elevation on the opposite tower for use as a foresight to eliminate all corrections due to curvature of the earth surface, errors in adjustment of instrument, etc. Telephones at the instrument platform, tower tops, and bases, with a special tie-line across the river, permitted intercommunication of information relative to deflections, instructions for adjusting the ropes, etc., between the instrumentmen and the workmen.

As has been stated, one rope only in each group was adjusted by instrument, the remaining ropes being grouped to as nearly the correct relation to the first as could be observed with the aid of telescopes. Throughout the entire operation of adjusting footbridge ropes, cross-heads with threaded tie-rods were used to connect the side-span and main-span sections and were kept tight to carry the stress of the rope if a jack should fail. The structural frames

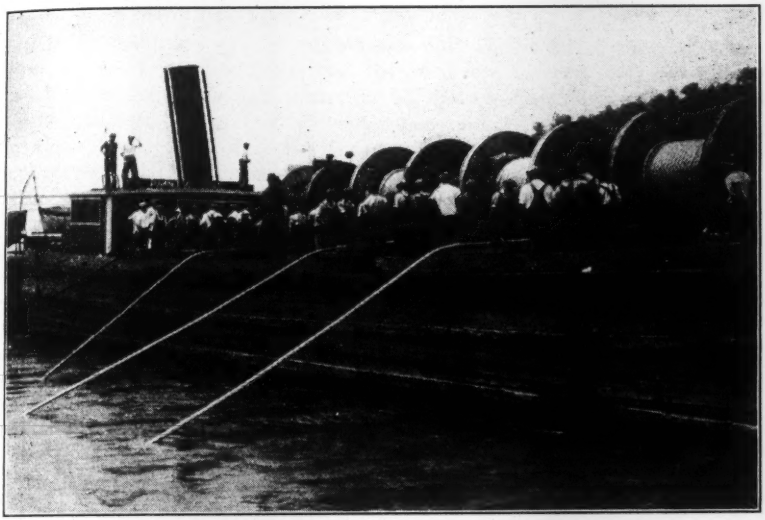

FIG. 8.—LAYING FOOTBRIDGE ROPES IN HUDSON RIVER.

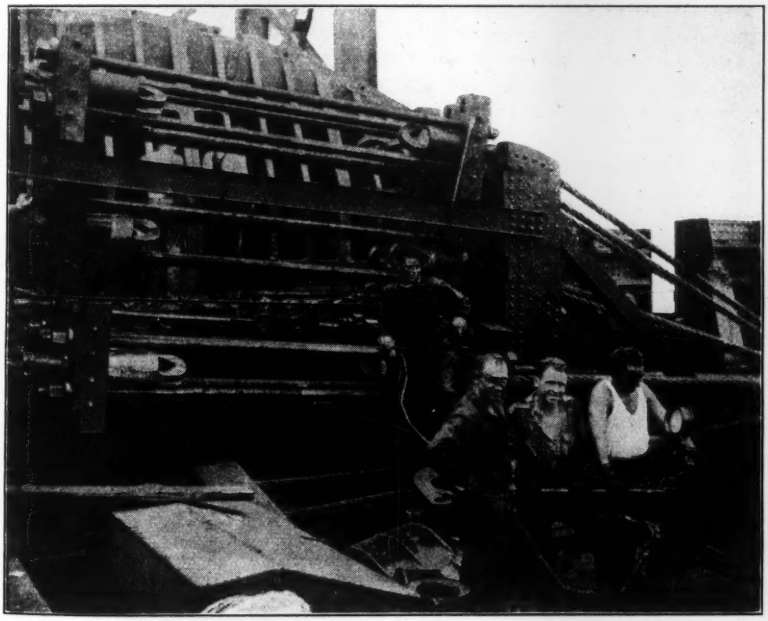

FIG. 9.—ADJUSTING ROPES IN TOWERS.

supporting the ropes were designed to permit separation of the individual ropes to provide clearance for these rods and also to carry only the unbalanced pull of the footbridge ropes; that is, the side-span ropes and the tower together resisted the pull of the main-span ropes. When the adjustments were completed, shims were left between the frame and the sockets of the rope sections of the main span.

Cable Saddles.—The anchorage saddles served not only to take care of the change in direction at the anchorages, but also as splay castings for the various strands composing the cables. Inasmuch as the center-to-center distance of the cables of each pair was considerably less at the saddles than at the eye-bar groups, it was necessary to set the saddles of each pair in planes inclined to each other as well as longitudinally, in order that their bases might be normal to the cable reactions. The saddles were also set with their axes at an angle to each other rather than parallel, as would be the case if there was no change in horizontal direction of the cables at this point. The saddles, which weighed approximately 21 tons each, rested upon segmental rollers which, in turn, rested upon 4-in. steel slabs. (See Fig. 10.)

The concrete of the anchorage structures was finished to the approximate planes for setting the slabs, allowance being made for about 2 in. of grout. Wooden templets designed to give the desired inclination of the slabs were used to set them to approximate position. Survey points were established on the center line between cables and on reference lines just riverward of the slabs. Computed offsets to these lines, together with elevations at each corner, were used in the final setting before grouting. The segmental rollers were then assembled on the slabs and blocked temporarily until the saddles were erected.

The anchorage saddles were then erected, slings being devised to suspend them at the desired angle. Wire-rope bridles and turnbuckles served to set and maintain the saddles in position until the first few sets of strands were in place and the resulting friction was sufficient to maintain them. This position was about $2\frac{1}{2}$ in. shoreward of normal, in order to allow for stretch in the back-stay due to the dead load of the structure.

After the footbridge ropes were erected, points were established on the grillages at the tower tops for roller and saddle erection. A careful procedure was followed to insure the correct alignment of the cable saddles. A point on the center line of the cable was established at the shore end of each grillage. A transit mounted on a trivet was then set up over these points and, with the points on the far tower as foresights, the center line of cable was established at the river end of each grillage.

This method of establishing the center line of cables on the grillages served to correct for any possible skew in the tower, which, however, was very slight. The rollers, spaced by end bars, were then placed in position on the slab and carefully retained in proper relation to allow for the motion of the saddle relative to the grillage.

Each cable saddle is made up of four separate castings, the heaviest weighing approximately 55 tons. The section nearest the shore span was

erected first. Fig. 11 is a view showing one stage of the procedure. The alignment was maintained by means of the transit sighting on center-line points established on the saddle sections. The saddle was then blocked in position approximately 2 ft shoreward of the transverse center line of the tower.

Segmental blocks were provided at both tower and anchorage saddles to restrain lateral motion of the strands in the upper half of the cables. In the case of the tower saddles, however, the final position of the strands was such that these blocks did not provide the desired support. Therefore, lead antimony was melted and poured between the blocks and the strands. This metal was adopted due to its low melting temperature and freedom from shrinkage on cooling.

Falsework at Top of Tower.—The erection equipment at the tops of the towers was assembled after the cable saddle castings were in place. It consisted primarily of a steel superstructure carrying a crane runway 51 ft above the top of the main steelwork as shown in Fig. 12. Two track girders extended the full width of the tower (transverse to the bridge axis). Each was supported by four pairs of braced columns, one pair on either side of the main cable saddles. Cross-bracing between the column rows stiffened the structure in a direction parallel to the center line of the bridge. Lateral stiffness was supplied to the track girders by trusses with horizontal bracing in the planes of the top and bottom flanges. Within the areas occupied by the cable saddles the runway was designed to withstand a load of 130 tons; and, over the area, between the two pairs of cables, a load of 15 tons. Cantilever brackets designed for a loading of 15 tons were attached to either end of the runways to permit the cranes to run out over the ends of the towers to hoist material from the ground. The falsework was also designed to support platforms over the main cable saddles which were required to support electrically driven machines, first used in erecting the footbridge decking and, later, in reeling up the footbridge ropes.

Footbridges.—Under conditions normally found in suspension bridges, the quantity of rope required for suspenders in the permanent structure exceeds that required for supporting the footbridges. On this bridge, however, the reverse of these conditions existed, so that it was essential to reduce to a minimum the weight of the footbridge decking and other miscellaneous dead load, in order that the quantity of excess rope required solely for supporting the footbridges might be reduced to a minimum. Therefore, the contractor resorted to steel framing partly covered with wood decking and partly with woven wire mesh as shown in Figs. 13 and 14. A central walkway, 5 ft wide between the cables, and two walkways, 3 ft 6 in. wide at either side, were provided. Woven wire mesh was used to cover the areas between the walkways directly under the positions to be occupied by the completed main cables.

For the main-span footbridges the steel framing was fabricated at the shop in sections consisting of 8-in. channel cross-beams spaced 12 ft back to back and joined by six longitudinal trusses made up of a bent-rod Warren web system welded to chord angles (see Section *B-B*, Fig 14). The trusses

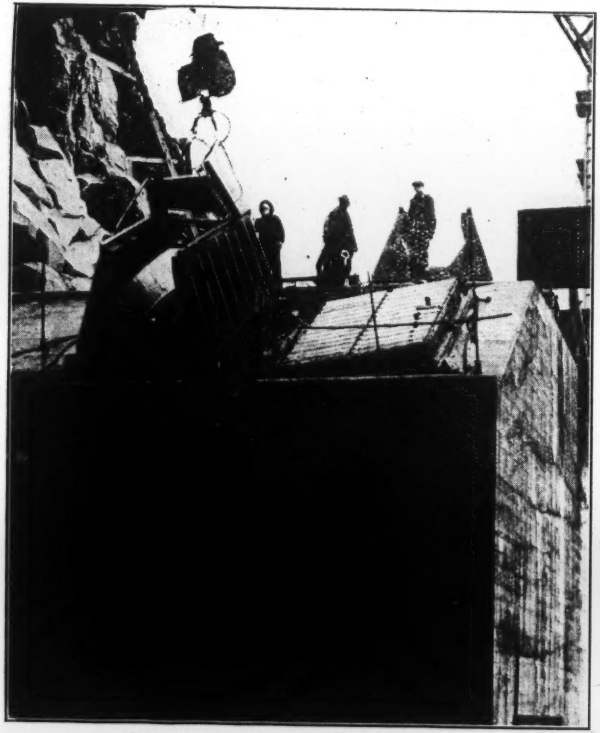

FIG. 10.—SETTING ANCHORAGE SADDLES.

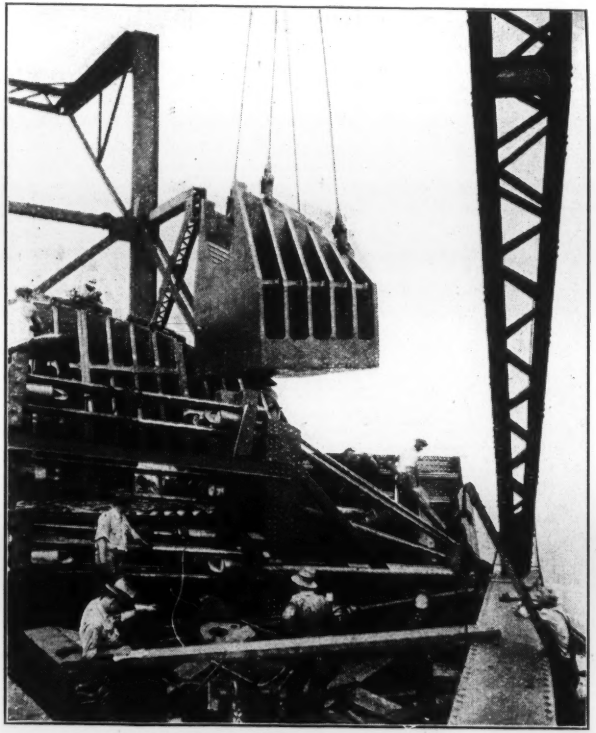

FIG. 11.—ERECTING TOWER SADDLE CASTINGS.

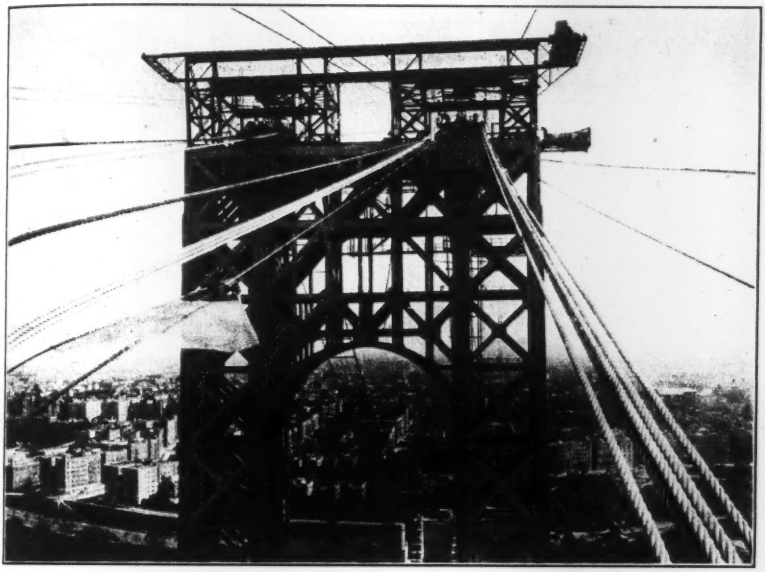

FIG. 12.—TOWER FALSEWORK.

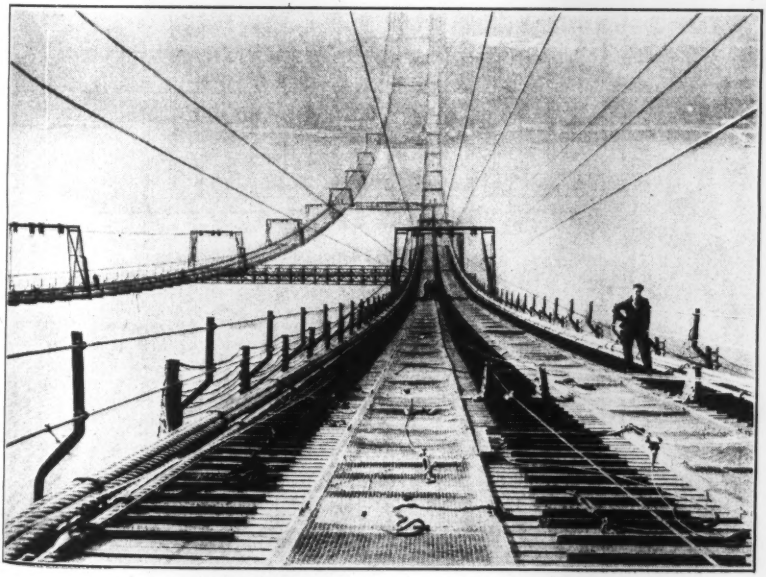

FIG. 13.—FOOTBRIDGES IN MAIN SPAN.

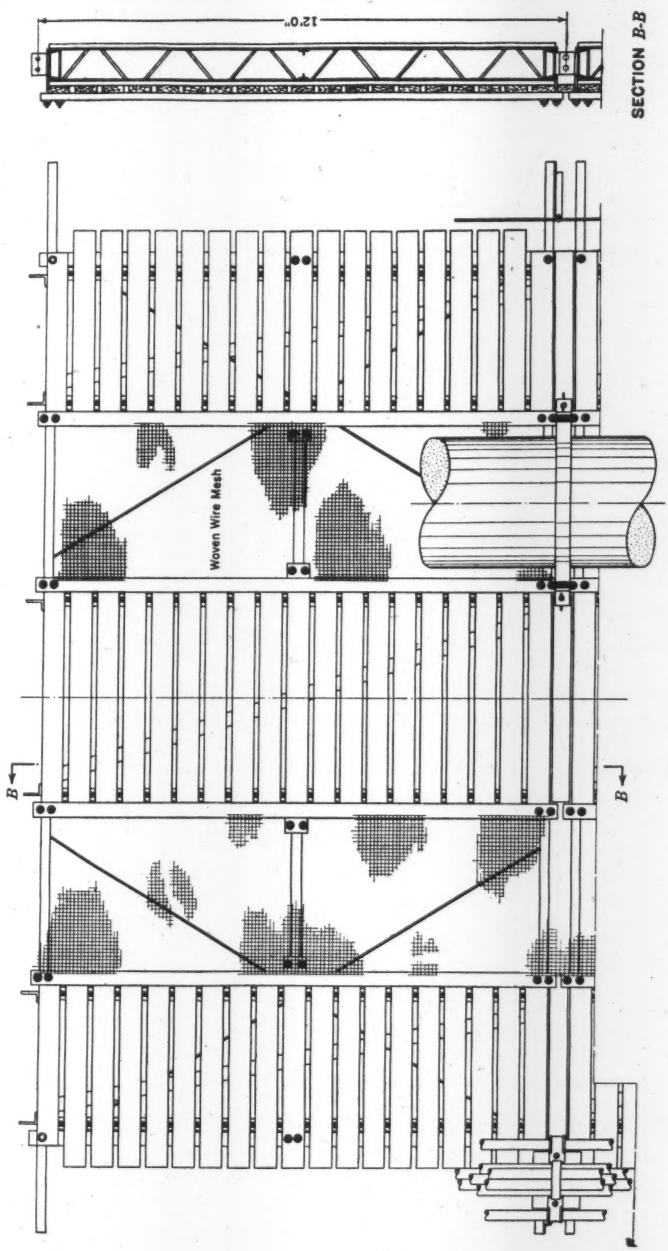

FIG. 14.—PLAN OF MAIN SPAN FOOTBRIDGE.

were spaced to support the edges of the three walkways and the entire unit was braced by diagonal tie-rods. The sections were bolted to the underside of the footbridge ropes by means of bent straps over the ropes. (See Fig. 15.)

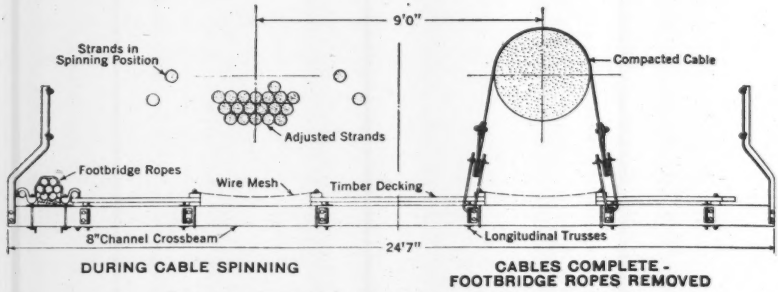

FIG. 15.—CROSS-SECTION OF MAIN SPAN FOOTBRIDGE.

The side-span footbridges were designed as a series of steps and landings because of the steepness of the slope. As in the main span, each footbridge provided for three walkways located to give access to the cables. The walkways were formed by units consisting of pairs of trusses designed so that the top chord and diagonals of the trusses could be used to support two flights of steps and two intervening landings. These units were supported on 10-in. pipe cross-beams spaced 30 ft apart and suspended from the footbridge rope groups by means of turnbuckle suspenders. The explanation for the suspension of the sections in the side span is that the contractor had intended originally to build the cables by the method of successive erection and had designed the anchorage for the footbridge ropes accordingly. Under the successive program the footbridges would have had to be lowered to clear the loaded position of the first pair of cables. To meet this condition, the contractor had designed the sections to be suspended from the footbridge ropes rather than support them on top of the ropes, as is done customarily. The plans for the equipment had progressed so far that, at the time decision was reached to construct all four cables simultaneously, the contractor preferred to utilize the original method of supporting the footbridges from the ropes. In the main span, however, the footbridge sections were brought into contact with the ropes instead of being suspended as in the side spans. Chemical fire extinguishers were installed in cages suspended at intervals beneath the footbridges to guard against the destruction of the footbridges by fire, as happened in the construction of the Williamsburg Bridge at New York, N. Y. As an added precaution, all timber was painted with fire retardant paint and sheet metal fire stops were placed under the decking at approximately 600-ft spacing.

After the falsework at the tops of the towers was completed and after the footbridge ropes had been adjusted, the footbridge decking was erected. This operation could not be carried out in the usual simple manner, by sliding the sections out from the towers over those previously placed, because

of the method adopted for supporting the decking. Different methods of erection also had to be used in the main and in the side spans.

For the main span, the footbridge sections were hoisted to the tops of the towers in skip cars, where they were loaded into carriages designed to run on the footbridge ropes in which they were transported to position for attachment to the rope groups.

The L-shaped steel frame shown in Fig. 16 was one of the skip cars for hoisting the sections of decking. It was guided by counterweighted skid ropes while being hoisted to the tower tops by the crane operating on the cantilever bracket of the runway. Fig 16 also shows the steel frame by which the car was brought to an inclined position at the base of the tower for loading. Steel links at the upper end of the car engaged steel plates riveted to the top of the tower, which allowed the car to be rotated through 90° to a horizontal position for attachment to pennant ropes hung from the end of the crane runway. When unloaded by the crane, the decking was transported over the cable saddles, deposited on the tower tops near the center line, and thence relayed, one section at a time, to hand-cars operating on the temporary platform shown in Fig. 17, which was attached to the river face of the tower below the level of the footbridges. The carriages riding on the rope groups were loaded by hoisting several sections into the opening provided for the purpose.

Footbridge sections were erected from the middle of the span toward the towers, with four carriages such as that shown in Fig. 18, two operating from each tower. Each carriage consisted of a rigid steel frame pin-connected at each corner to a two-wheeled truck. The wheels were made of maplewood restrained between steel plates, with the rims shaped so as to conform to the grouping of the footbridge ropes. Wooden blocks enclosing the group of ropes were mounted on arms in front, and in back, of the wheels to hold the ropes in their correct relation with each other. The blocks were divided on their horizontal axis to facilitate easy removal and re-attachment. This was a necessary precaution to permit clearing the erected sections of decking.

An under-slung steel cage was hung from the frame by means of a single pin at either side, an arrangement which permitted the cage to hang vertically regardless of the varying inclination of the frame corresponding to the slope of the ropes. The cage was designed to form two longitudinal working platforms, one above the other on each side, and two transverse platforms over each other across each end. Easy access to the various platforms was provided by ladders in each corner. Hand winches on the lower transverse working platforms of the cage were used to hoist the footbridge sections into the cage. These winches were arranged so that they could also be driven by pneumatic drill machines to speed up the operation of hoisting the sections.

The carriage was then shifted along the ropes, propelled by an endless wire rope (later used as a hauling rope) passing around idler sheaves at either anchorage and around an electrically driven spool mounted on the temporary platform above the main cable saddles. When in position for erection

the top section of the pile was raised into contact with the under side of the ropes. The end adjacent to the section previously erected was permanently fastened to it with plate and angle clips and then bolted to the footbridge ropes. The other end was temporarily supported by hangers until the carriage was shifted the length of the footbridge section in order to bring the next section into position. Four hand winches mounted on the upper longitudinal working platforms were used to raise the sections.

A minor change in the original method of rigging the rope for propelling the carriage was found necessary as the erection progressed. This change consisted in restraining the opposite part of the propelling rope in a sheave on the carriage to limit its variation in sag, in order to make the carriage move more uniformly toward the middle or flatter part of the span. Aside from this, the carriages functioned very well, and the erection of the main-span footbridges was completed in eight working days. Five steel truss cross-bridges connected the north and south footbridges in the main span. These were hoisted directly from barges and bolted to the footbridge ropes.

The side-span decking was erected simultaneously with that of the main span in order to balance the pull of the ropes on the towers. The units forming this decking were also erected (from the anchorage toward the towers) by means of carriages operating on two ropes stretched above, and approximately parallel to, the footbridge ropes. Before these units could be erected, however, the pipe cross-beams forming their support had to be placed and suspended from the footbridge ropes. The ends of the pipes were equipped with steel castings that engaged a pair of 2-in. wire ropes stretched from the top of the tower to the anchorage, which were used to support the pipe beams temporarily. These ropes were anchored through counterweighted wire-rope falls to prevent them from participating in the load of the footbridge ropes. The pipe beams were assembled on the 2-in. ropes at the anchorage and were pulled up toward the towers, spacer ropes serving to fix and maintain the correct distance between beams. They were then clamped to the ropes with a wedge locking device and the suspenders connecting them to the footbridge ropes were erected, the workmen making use of a hanging platform suspended from the erection carriage.

With the suspenders in place the decking was transported by the carriages from the anchorage to position in the footbridges as shown in Fig. 19. Three units, comprising a complete panel, were carried on each trip, chain blocks being used to raise or lower them as necessary.

Storm System.—It has been the practice to stiffen footbridges against excessive deflections resulting from severe winds or storms. This is usually done by stretching wire ropes between the tower bases. These ropes are arched upward nearly to meet the footbridges at mid-span by means of vertical hangers attached to the footbridge ropes. Additional stiffness adjacent to the towers is sometimes obtained by the use of diagonal guy ropes from one or more points on the footbridge ropes to the towers at approximately the floor level. Due to the unprecedented span of the George Washington Bridge, the contractor was unwilling to rely upon this system of storm ropes. In an effort to find a more adequate arrangement, a scale model of the footbridges

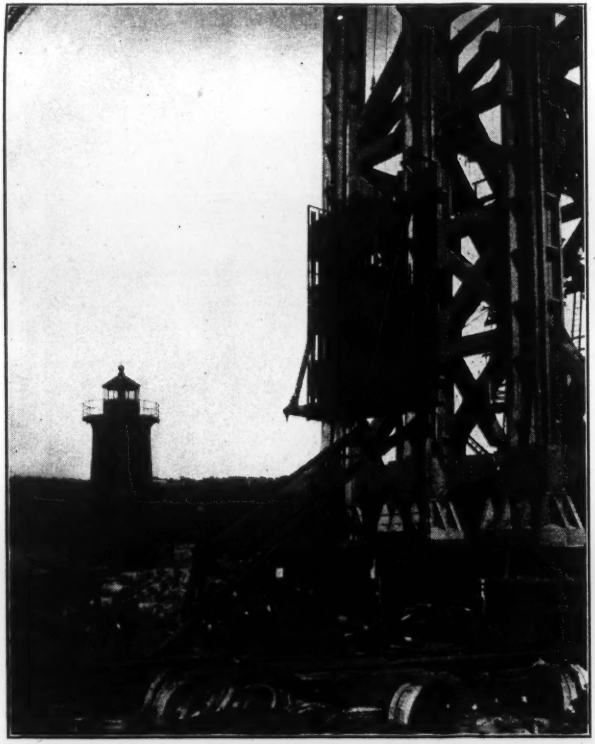

FIG. 16.—SKIP FOR HOISTING SECTIONS.

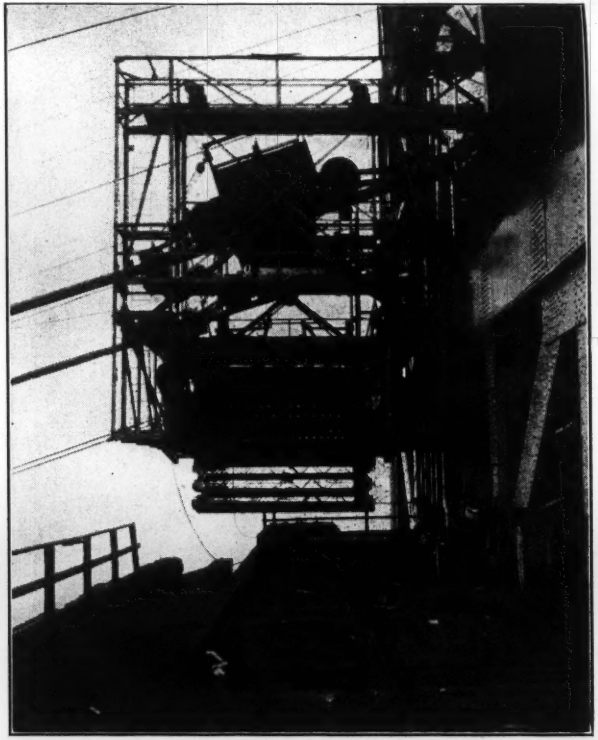

FIG. 17.—LOADING MAIN SPAN CARRIAGE.

University of Michigan

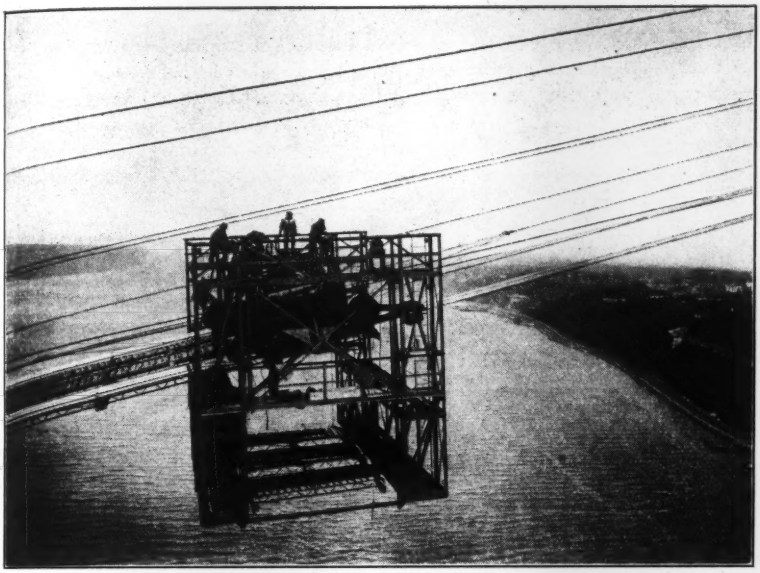

FIG. 18.—MAIN SPAN CARRIAGE.

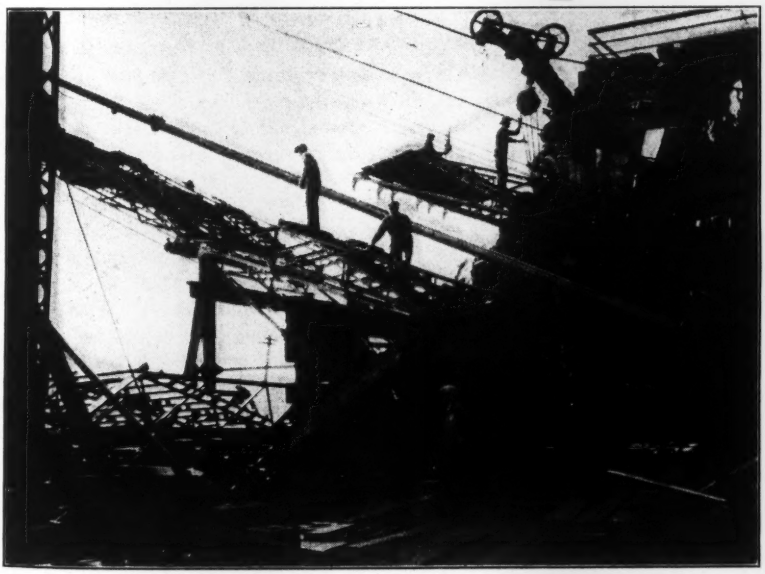

FIG. 19.—ERECTING SIDE SPAN DECKING.

was constructed upon which experiments were made with various systems of ropes. The one adopted as being the most effective in dampening both vertical and lateral deflections contained the salient features of the systems commonly used, with certain additions.

All the ropes were of high-strength steel, $\frac{1}{2}$ in. in diameter, except as indicated in Fig. 20. The arched ropes, two for each footbridge, were attached

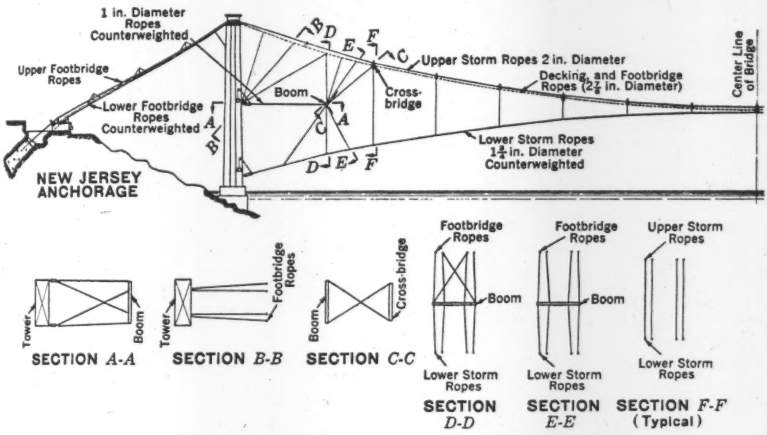

FIG. 20.—CROSS-SECTIONS OF STORM SYSTEM.

at one end by means of counterweighted wire-rope falls. The hanger ropes were attached to ropes suspended above the footbridge ropes and the footbridges were connected to the hanger ropes.

Latticed angle struts, about 42 in. square and 140 ft long, were suspended beneath the footbridges at either end of the main span, 310 ft from the towers and approximately 300 ft above the water. Four sets of diagonal ropes connected each strut to three points on the footbridge ropes and to two points on the lower storm ropes. The struts were also braced laterally with ropes to the footbridge ropes and to the towers. Those connected to the tower passed through sheaves at the towers and back to attachments on the struts, the sheaves, in turn, being fastened to the tower by means of counterweighted wire-rope falls. The counterweights at the various connections to the towers were used to maintain relatively constant stresses in the storm system for any variations in temperature or loading of the footbridges. In actual use the storm system proved very effective, no deflections being noticeable in the footbridges, except during a few periods of severe winds.

At various points on the footbridges and storm system and also on the tower tops electric lights were installed for the protection of aerial navigation.

Method of Spinning Parallel Wire Cables.—The basic principles of spinning parallel wire cables have become well known by reason of the large number of suspension bridges built in recent years. They were used more than seventy-five years ago by the late John A. Roebling, M. Am. Soc. C. E.,

in constructing the Niagara Suspension Bridge and, again, by his son in building the well-known Brooklyn Bridge. Essentially the same principles, with certain refinements, have been used ever since.

For the benefit of those who may be unfamiliar with the construction of parallel wire cables, the following brief description is given.

As its name implies, a parallel wire cable is simply a bundle of parallel wires attached at either end of the bridge to steelwork built into structures, known as anchorages. The wires making up the cable are divided, for purposes of construction, into several units or groups called "strands," which lose their identity in the completed cable except at either anchorage where the various strands splay out to separate connections to eye-bar chains and

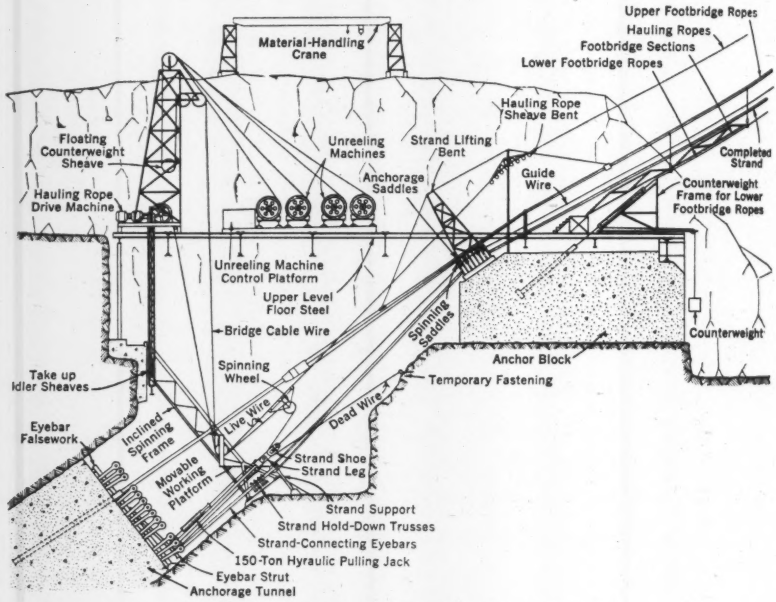

FIG. 21.—SECTION AT NEW JERSEY ANCHORAGE.

girders forming the anchorage steelwork. The wire in any one strand is continuous and is looped around a horseshoe-shaped steel casting or "strand shoe" at either end, which is inserted between a pair of eye-bars and pin-connected to them. The entire arrangement is shown diagrammatically in Fig. 21.

The strands are usually constructed in individual supports or saddles outside the main cable saddles with the strand shoes temporarily connected to the eye-bars by means of "strand legs," which are steel castings constructed so that one face of the strand shoe is exposed to permit the wire to be looped around it. A strand is spun by first taking the exposed end of a reel of wire

mounted at the anchorage and attaching it to any convenient point in front of the strand shoe. The wire is laid around the strand shoe and a loop of wire is formed, one part leading to the strand shoe and the other to the reel. This loop is then progressively lengthened by pulling wire from the reel, until it reaches the other anchorage where it is placed around the strand shoe at that point. The wire leading to the reel is then placed around the near strand shoe again, forming a second loop which is similarly lengthened and placed around the far strand shoe. This operation, called "spinning," is repeated until the desired number of loops or wires have been placed in the strand. The wire leading to the reel is then cut and spliced to the end previously fastened in front of the strand shoe, thus making the wire continuous. The loops of wire are pulled from anchorage to anchorage by placing them around a deep flanged wheel known as a "spinning wheel" which is attached to an endless tramway or "hauling rope.

To speed up the spinning operation, a reel of wire is usually mounted at the opposite anchorage and wire is pulled out for a second strand on the return trips of the spinning wheel. A second spinning wheel attached to the other part of the hauling rope and traveling in the opposite direction to the first wheel is used to pull out wire for two more strands.

The wires are adjusted individually to the desired sag in each of the three spans. This operation is as follows: As soon as the spinning wheel has passed over the first tower, the wire from the strand shoe, or the "dead" wire, is pulled through the strand saddles at the tower until it hangs at the correct sag in the side span. It is then clamped at the tower to prevent slipping. After the spinning wheel passes over the second tower the same wire is adjusted in the main span and clamped at the second tower. When the loop has been placed around the far strand shoe, the wire is rendered around it until the "dead" wire hangs at the correct sag in the far side span and is clamped at that point. The "live" wire, or the wire leading to the reel, is then similarly and successively adjusted in the far side span, main span, and near side span, so that all slack in the wire is carried ahead from span to span and ultimately into the next loop. After both wires of a loop are adjusted, they are removed from the clamps.

The sags for adjusting the wires are fixed by "guide"-wires which are measured to length in advance by adjusting them to hang at certain sags established by instrument in a manner similar to that used in adjusting the footbridge ropes. After a small group of wires are in place in each strand, the guide-wires are removed, and the remaining wires are adjusted to the group. The lengths of guide-wires are changed according to the computed variations in the lengths of the strands and are used at the start of each set of strands without having to be set every time by instrument.

When a set of four strands has been spun, the wires in each strand are bunched together with a nominal seizing. The strand is then transferred from the strand saddles to final position in the cable saddles, the strand shoes at the same time being removed from the strand legs and permanently connected to the eye-bars (see Fig. 15).

After all the strands have been completed and placed in their final position in the main saddles, the cables are compacted or squeezed in the spans between the saddles. The squeezing breaks up the circular strands and fills in the voids between them to form a single compact circular cable as shown in Fig. 15.

The number of strands to be used in constructing a cable is generally fixed so that they can be built up in layers to form a hexagonal cross-section prior to squeezing. Thus 7, 19, 37, 61, etc., may be used. The required cross-sectional area of the cable combined with the limiting size of strand determines which multiple to use. As has been previously mentioned, sixty-one strands, each containing 434 No. 6 wires were used to make up each of the four cables for the George Washington Bridge.

Adjustment of Guide-Wires.—The guide-wires were set just prior to the completion of the erection of cable-spinning equipment. Four wires for each cable were pulled over the footbridges by hand. The galvanizing had been omitted from these wires to make them easily distinguishable from the cable wire. As has already been mentioned, these wires had to be adjusted to the correct sags as established by instrument.

The sags are usually measured in both main and side spans. However, in this case, due to the relatively short side spans as compared to the main span, it was decided to follow a procedure that would require instrument observations in the main span only. The wire to be measured was mounted on 1-in. steel rollers, spaced 1 ft apart on a steel strap laid in the center groove in each tower saddle. When so placed, the wire immediately adjusted itself to balance the tension on both sides of the saddle, in such a way that fixing the sag in the main span would automatically fix the sags in the side spans. The total length of guide-wire could thus be measured very accurately. The wire was then pulled in or slacked off at one anchorage the amount necessary to secure the desired sag in the main span, allowance being made for variation in temperature above or below the normal, which was 50° F. With this method of adjustment, tower deflections could be neglected since they had practically no effect on the sag because of the free movement of the wire on the rollers. Observations were made by means of a level instrument set up on the tower, sighting on an inverted target rod lowered through the footbridge decking at mid-span. Only one wire of each set was adjusted by instrument, the remaining three wires being made to correspond to the first.

Improvements in Cable-Spinning Equipment.—Several new features were devised for spinning the cables of the George Washington Bridge. These functioned very well and made it possible to increase the rate of spinning cable wire beyond anything previously accomplished.

An improved method of mounting the reels of cable wire at the anchorages was introduced. It had been the practice simply to mount the reels in a rack, relying upon the pull of the spinning wheel to rotate the reel. Manually operated brakes served to slow down or to stop the reel when necessary. The inertia of the reels, particularly when filled with wire, would cause considerable surging and whipping of the live wire along the footbridges whenever the speed of the spinning wheel varied.

To avoid these difficulties, the reels were mounted on machines that rotated them in a vertical plane at the speed required to unreel the wire as pulled out by the spinning wheel. A specially designed reel was used, consisting simply of a steel plate cylinder, 6 ft in diameter, with bent channel flanges riveted to either end. The reels each had a capacity of about 8 tons of cable wire, a continuous length of about 30 miles. They were slipped on and off an overhanging cylindrical member of the machine with the aid of hand-cars. Six telescoping arms extending out from the cylinder as shown in Fig. 22 served to raise the reels clear of the hand-cars and lock them in place as shown in Fig. 23.

Eight unreeling machines, two for each cable, were required at each anchorage. The driving power had to be capable of rotating the machine at varying speeds up to approximately 75 rpm, and also capable of changing from one limit to the other of this range within a few seconds. Hydraulic power was adopted as being most suitable for these conditions. It consisted of an electrically driven hydraulic pump, the output of which could be varied in amount and also changed in direction by means of a manually operated regulator. The pump was connected with high-pressure pipes to a hydraulic motor geared to the unreeling machine. One 100-hp electric motor was used to drive a set of four pumps serving the four unreeling machines for a pair of cables at each anchorage. Only two of the four machines, of course, were ever in operation at the same time. On the New York side the machines were set up on top of the anchorage block, behind the eye-bar pits. Those on the New Jersey side were set up on the floor steel directly over the eye-bar pits. (See Fig. 21.)

As an indication to the operator of the speed of the unreeling machine in relation to the speed of the spinning wheel, the wire from the reel was run through a compensating sheave tower where momentary variations between rates of unreeling and pulling out were absorbed. This also maintained practically uniform tension in the "live" wire, eliminating all tendency for it to whip along the footbridges. Two deflecting sheaves were mounted in line at the top of a structural steel tower, as shown in Figs. 7 and 21. The wire coming from the reel was led over one deflecting sheave, down under a floating sheave, and then up over the other deflecting sheave. The operator of the reel machine varied its speed to maintain the floating sheave as nearly as possible at the mid-point of its range of travel.

The wire of one reel was spliced to that of the next with a sleeve coupling pressed into place with a three-jaw hydraulic press. This splice was identical to that used in the shop for the manufactured lengths of wire.

Another quite important departure from customary practice was the manner in which the spinning wheel was attached to the hauling ropes which required a new type of hauling rope support. The spinning wheel was mounted on the side of the rigid, triangular, structural steel frame shown in Fig. 24, the two upper corners of which were supported by pin-connected steel-plate links from two steel plates parallel to, and directly under, the hauling rope and attached to the rope by two steel straps. These straps

were bent around a bronze collar clamped to the rope. The collar permitted the rope to rotate within the straps and eliminated any tendency of the lay to tighten or loosen at these points. The pull of the hauling rope was transmitted to the spinning wheel by means of a third plate similarly attached to the rope directly over the wheel, and pin-connected to the frame. The pin-hole in the plate was slotted in a direction normal to the hauling rope in order to provide for movement between it and the frame while the rope is passing over the sheaves. The intervals of rope between the straps were filled with collars of the same diameter as the straps, and tapered collars were placed at each end, to create smoother operation over the sheaves.

An entirely new type of sheave was required for supporting the hauling ropes, because the attachment to the spinning-wheel frame was made under the rope rather than on one side as had been the former practice. The sheave designed to meet this condition consisted of two independent wheels mounted opposite to, and at an angle with, each other, such that the distance between their flanges was a minimum at the top. The clearance between the flanges was fixed just so as to allow the straps supporting the spinning-wheel frame to pass. Pins attached to the axles of the wheels and projecting through spiral slots cut in the caps clamping the axles to a cast-steel yoke, served to adjust this clearance quickly. The flanges of the wheels were machined to form a groove centered directly over the gap between them, the lower part of which was of a diameter such as to fit the hauling rope and the upper part of a larger diameter to fit the straps and collars of the spinning-wheel attachment. A series of these sheaves were placed in an arc at the towers and over the anchorage saddles, to take care of the change in direction of the hauling ropes at these points. At the steel bents, which were spaced at about 225-ft intervals along the footbridges, idler sheaves were also mounted above, and in line with, the support sheaves. The tension of the hauling ropes was kept sufficient to maintain a slight upward reaction at these points, except during the passage of the wheel, thus reducing to a minimum the wear on the hauling rope resulting from the partial support of the double-wheel sheaves.

The hauling ropes passed around idler sheaves at the New York anchorage. The driving equipment and the arrangement for maintaining tension in the rope were located at the New Jersey anchorage where the ropes passed around driving drums and around adjustable idler sheaves directly below the drums. Adjustment was secured by wire line falls pulling down on the idler sheaves.

The ropes were driven by means of 100-hp reversible electric motors through "Philadelphia" reduction gear units. At first, the maximum speed was set at approximately 620 lin ft per min. It was later increased to 675 lin ft per min, or 70% greater than had been attained previously on similar work.

Movable working platforms supported over the anchorage eye-bars just behind the strand shoes (shown in Fig. 7) were also an important feature. As the sets of strands were completed, these platforms were moved up on a

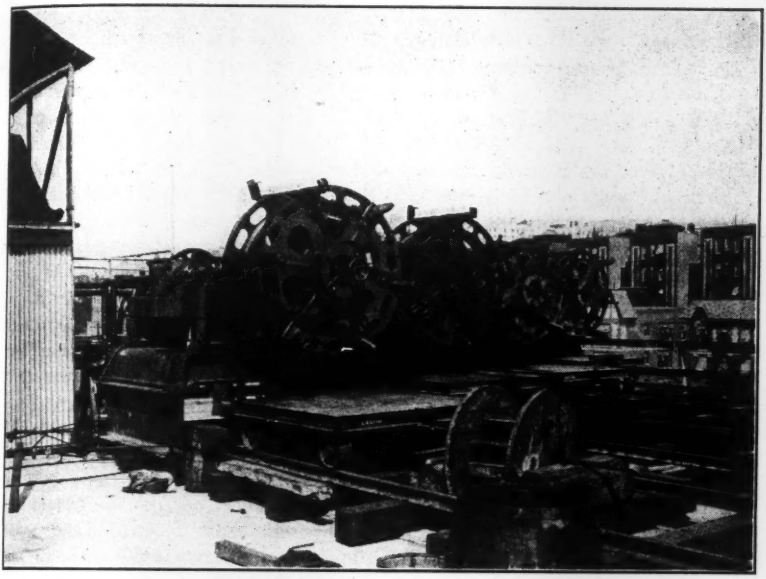

FIG. 22.—VIEW OF UNREELING MACHINE.

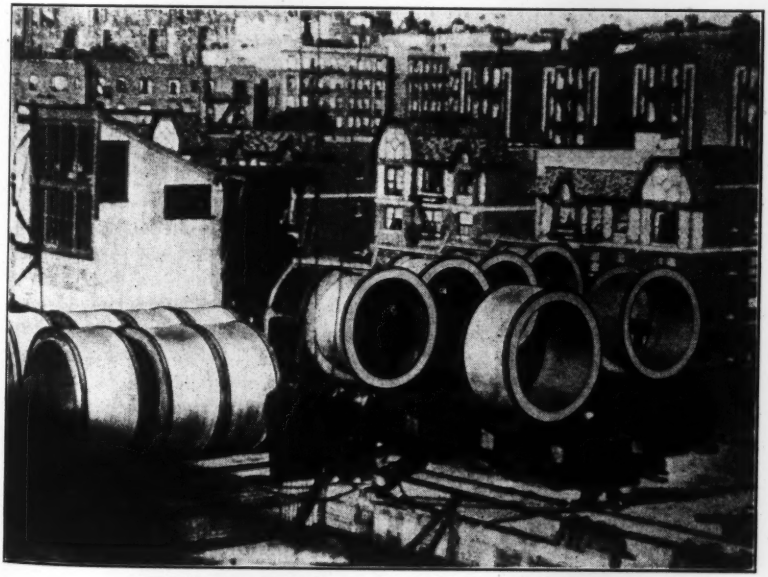

FIG. 23.—REELS MOUNTED ON UNREELING MACHINES.

pair of inclined trusses erected on either side of the eye-bars. The hauling ropes and the live wires from the compensating towers passed around deflection sheaves mounted on these platforms, since it was necessary to bring the spinning wheels and the wire loops within easy reach of the workmen.

A dispatcher on the New Jersey side maintained complete control of the operation of the spinning wheels by means of an electric signaling system. Push-buttons connected to signal bells at each anchorage, were placed on the working platforms at the eye-bars, at each tower, and at every hauling rope bent along the footbridges. Each of these buttons was also connected to an annunciator board in the dispatcher's house. The workmen at each anchorage would signal when they had their loop of wire in place around the spinning wheel and after receiving the signal from both anchorages the dispatcher, in turn, would signal the hauling rope operator by buzzer to start the wheels. If, for any reason, it was necessary to stop the wheels at some point along the footbridges, the men could so signal the operator with the push-button nearest that point. The dispatcher knew immediately at what point the wheels had been stopped and by telephone could determine the reason. When the trouble had been corrected, a starting signal over the same push-button notified the dispatcher to signal the operator to start the wheels. This arrangement eliminated much lost time in locating the source of any trouble and also the likelihood of accidents resulting from some one signaling to start the wheels from some point other than that at which they had been stopped.

As previously mentioned, the wires were adjusted individually to hang at the correct sag in each of the three spans. This is usually done by attaching to the wire beyond the supports through which the wire is to be rendered, a lineman's wire-gripping tool commonly known as a "come-along." The "come-along" is connected, in turn, to a light manila rope falls with which workmen stationed at these points pull up or slack off on the wire as signaled by an adjuster stationed at the mid-point of the span in which the wire is being adjusted. Ordinarily, this method is entirely adequate but in the case of the George Washington Bridge it had several disadvantages. In order to adjust the wires it was necessary to pull them up in tension to approximately 600 lb., a greater force than could easily be applied single-handed without making the workman travel excessive distances in adding more parts to the falls. Furthermore, the length of the main span was too great to permit visual signaling from mid-span to the towers. Therefore, mechanical units at the "come-alongs," controlled by the adjusters in the spans, were adopted. The units, one set for each cable, were installed on either side of both towers and at each anchorage. Each unit consisted of an endless rope that passed around two sheaves mounted along the footbridge and thence down to an electrically driven drum placed beneath. Pennant lines attached to the rope and to the "come-alongs" thus performed the same function as the block and falls. The electric motor driving the unit was reversible and was controlled either by the man at the "come-along" or by the adjuster in the span.

A system of standing electric light signals was also adopted to indicate when the wires were ready to be adjusted and when the adjustment had been completed. For instance, when a dead wire was adjusted in the near side span and clamped, the men at the tower would turn on a pair of lights, one at the tower and the other at the middle of the main span. When the spinning wheel had passed over the far tower and the men had the "come-along" in position on the wire, they would turn on a pair of lights similarly located. Both lights burning at mid-span was a signal to the adjuster to proceed with the adjustment of the wire. When completed to his satisfaction, he would turn out both sets of lights which served as a signal to the men at the far tower to clamp the wire and remove the "come-along." Side-span adjustments were made in an identical manner. One set of lights in each span served for the wires in the pair of strands pulled out by one spinning wheel and an identical set served for the wires of the other pair of strands.

As an aid to the adjusters in the main span, men were stationed at the quarter-points to signal to the adjuster the positions of the wire at those points. This precaution was necessary in order to adjust, properly, such wires as would not hang symmetrically due to slight non-uniformity of weight, wind, conditions, etc. Later, it was found that results just as satisfactory could be obtained more quickly by steadying the wire at the quarter-points at the correct relation to the group, thus reducing the effective span length for the adjustment to about one-half.

Extremely accurate adjustments were obtained by this method. When a strand was completed it was customary to remove all banding throughout its length, thus allowing all wires of the group to hang freely for final inspection of adjustment. It was not at all unusual to find all the wires in the main span hanging within a range of 1 ft. Wires hanging decidedly above or below the group were cut and lengthened or shortened as necessary and spliced with threaded sleeve couplings.

Seizing, Placing, and Adjusting the Strands.—The strands were compacted into their circular form approximately 4½ in. in diameter by means of specially designed tongs so constructed that they would lock automatically when a certain diameter had been reached. They were easily adjusted to accommodate slight variations in strand diameter. The strands were compacted at approximately 5-ft intervals and were seized with cotton bands having metal clips. Another type of seizing, consisting of several wraps of cloth tape treated with an adhesive compound, was adopted for the later strands. It is the opinion of the writers that neither of these types of seizing was entirely satisfactory as it is believed that they did not break during the cable-squeezing operation to allow the wires to become fully re-adjusted so as to fill the voids. This may be partly the cause of the difficulty encountered in squeezing the cables, as mentioned later. This belief was strengthened upon observing the action of these bands while squeezing a model section of cable to check the calculated cable diameter.

The parts of the strand that would be placed in the tower and anchorage saddles were seized with a series of wire wrappings of several turns each

in the spaces between the strand saddles. At the anchorage the two halves of each strand were drawn together 24 ft from the strand shoe and seized for a length of 12 in. with $\frac{1}{8}$-in. stranded wire.

Balance beams such as that shown in Fig. 24 were used to lift the strands from the spinning position in the strand saddles to permanent position in the cable saddles. At the towers they were built in the form of girders with flanges curved so as to be approximately parallel to the curve of the strands passing over the tower. The strand to be lifted was attached to the bottom flange by a series of $\frac{3}{8}$-in. wire rope grommets, made up by hand from a single length of wire strand. These grommets were placed under the strand, and their ends were looped over small cast shoes attached to the lower end of adjusting screws on opposite sides of the balance beam. During the operation of placing the grommets the balance beam was supported above the strand by wooden blocks resting on the strand saddles so that the grommets could be adjusted to nearly uniform tension.

The strand was then lifted by two sets of 12-part falls hung from either end of the crane, and was shifted laterally and lowered into place in the tower saddle as shown in Fig. 25. This operation was carried on at one tower at a time to reduce the hazard of a strand tearing loose from the balance beam. While being lifted at the tower, the strand was similarly lifted at the adjacent anchorage saddle by a shorter balance beam attached to a 60-ton hydraulic pulling jack which was hung from a carriage rolling on a pair of I-beams. This arrangement is shown in Fig. 26. The beams were pivot-connected at each end to the sides of a steel bent erected over the saddles. The carriage support permitted the strand to be shifted laterally from spinning position to the final position, and the pivoting of the beams eliminated eccentric pull on the carriage during the operation of pulling back the strand to make the permanent connection at the strand shoe.

Before lowering the strand into position in the saddle, steel fillers shaped to fit around the under-half of the strand were inserted between the parts of each grommet. The fillers were a little thicker than the grommets. Strips of thin sheet metal were placed above and below the fillers to facilitate the withdrawal of both grommets and fillers from under the strand. The strips were then pulled out if possible; if not, they were cut off and left in place.

The first strand placed in the main saddles for each pair of cables was adjusted to hang at the correct sags established by instrument. In the side spans this was done according to the method described for adjusting the footbridge ropes and in the main span as described for adjusting the guide-wires. The first strand of the second cable of each pair was adjusted to hang level with the strand in the first cable which had been adjusted by instrument. The remaining strands of all cables were then adjusted to their proper relation with the strands already in place.

As first placed in the saddles, the strands were set to hang approximately 1 in. or 2 in. high in each of the spans, leaving the grommets under the strands at the anchorage saddles. Observations were made early the following morning when the strands were more nearly at uniform temperature, to determine the amounts by which the strands were too high. The strands

were then rendered through the saddles during the day the amounts necessary to lower them to their correct positions. These amounts were calculated from the known sag ratios, allowances being made for the effect of removing the grommets at the anchorage saddles. The strands were easily rendered through the tower saddles by supporting them with chain falls from the bents that held the hauling ropes in the side spans, and slacking off at the strand shoe. They were always held clear of the anchorage saddles during this operation. The strand adjustments in the main span were usually divided between the two sides.

During the operations of spinning and adjusting, the tower saddles were blocked in such a position that the horizontal component of the adjusted strands in the main span equalled the horizontal component of the adjusted strands in the side span at an assumed normal temperature. Under such condition the actual tension in the side spans was approximately 13% greater than that at the main span side of the saddle, and a friction of 13% between the strands and the saddle casting was required to restrain the strands against slipping toward the side span.

Comparatively slight changes in temperature affected considerable changes in the relative tensions in the strand in the tower saddle as between the main span and side span. The sag in the main span, approximately 300 ft, was such that shortening of the wire due to decrease in temperature had little effect on the stress in the strand at the tower. However, the sags in the side spans were so slight (9 ft in one case and 11 ft in the other), that even slight changes in temperature made quite material changes in stress in the strands on the shoreward side of the tower. This condition required special consideration during the adjustment of the first few strands because of the relative stiffness of the footbridge system and the tower, which resisted the comparatively small unbalanced forces exerted on it by the strands.

At the time of adjusting the first strand in the side spans, the temperature had dropped considerably below that anticipated when the saddles were located on the tower tops. This resulted in unbalancing the side-span and main-span tensions sufficiently to overcome the friction, causing the strand to slip through the saddles into the side spans. This condition was met by jacking the saddles farther shoreward, after which the strands could be readily retained in adjusted position. The friction was also increased by removing the shop coat of paint that had been placed on the machined surface of the saddles. As additional insurance against further slipping, the contractor also attached clamps at the main-span side of the saddle to the two lower layers of strands. These clamps were blocked against the saddles.

The strands were adjusted at the anchorages by means of 150-ton hydraulic pulling jacks as indicated in Fig. 27. As shown in Fig. 21 the jack completed one leg of a two-part rope-and-link tackle that was attached at either end to the strand leg and passed around a split sheave placed on the pin at the lower ends of the strand-connecting eye-bars. Cross-heads and side bars by-passed the jack so that it could be extended for a second pull or removed entirely without dismantling all the tackle. Shims were kept inserted between

FIG. 24.—BALANCE BEAM AT TOWER (ATTACHED TO STRAND BUT NOT LIFTED).

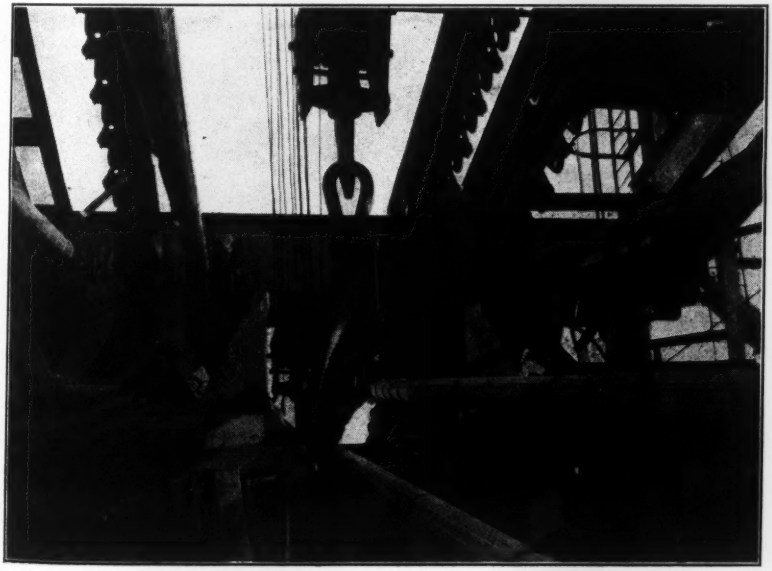

FIG. 25.—BALANCE BEAM PLACING STRAND IN SADDLE.

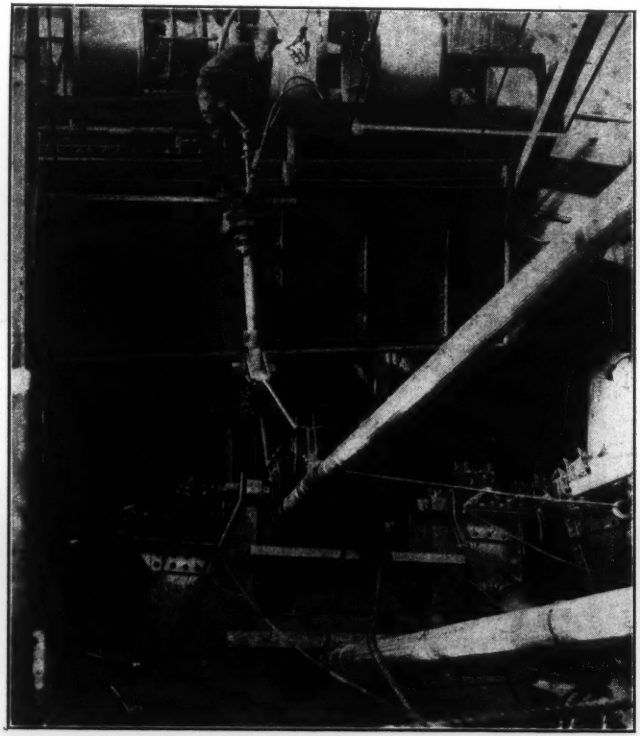

FIG. 26.—BALANCE BEAM AT ANCHORAGE.

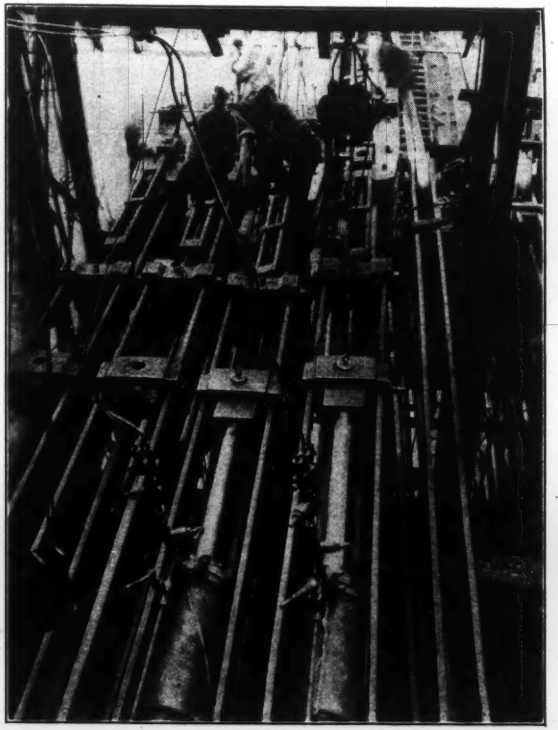

FIG. 27.—HYDRAULIC JACKS AT ANCHORAGE.

the lower cross-head and the adjustable lugs pinned to the side bars to prevent more than a limited movement of the strand in case the jack should fail.

In the past it has been the experience of engineers in constructing large parallel wire cables, that as the upper layers of strands are set in place and adjusted, unequal temperature conditions caused by the sun shining on the top layers cause the strands to lengthen and force their way down between the strands in the layers below. Frequently, the strands become so wedged between those below that they will not resume their normal relation when temperatures again become uniform. To overcome this difficulty, strand retainers, consisting of wood-lined hexagonal steel frames, were attached to the adjusted strands at seven points in the main span and at the middle of the side spans. The members forming the upper parts of the frame were built in sections which were bolted in place as the upper layers of strands were spun and adjusted. The frames were attached to the two bottom layers of strands by means of thin steel straps looped around the strands and tightened with threaded rod extensions projecting through the bottom of the frame.

A peculiar condition resulted from the use of this method of attaching the retainers. A twist developed in the main-span cables, causing the group of strands to rotate in one direction in one-half the span and in the opposite direction in the other half. This condition was most pronounced in the outside cables, which rotated in opposite directions. The twisting finally became so severe as to hamper the strand adjustments seriously. It proved to be due to uneven temperature conditions which caused a longitudinal shifting of the strands in relation to each other. The "retainers" did not prevent the strands from shifting, but did prevent them from returning to their normal position. This was finally corrected by simply loosening the steel straps attaching the frames to the strands.

Unusual Construction Details Required at Anchorages.—Several features in the design of the steel for the anchorage required special consideration in the construction of the cables. For economy, the steelwork in the New Jersey anchorage was designed with a change in alignment between the strands and their connecting eye-bars, with the eye-bars converging toward a point considerably farther riverward than that of the strands. During the spinning operation this condition and the temporary positions of strands in spinning saddles outside the cable saddles tended to create excessive bending stresses in the embedded eye-bars at their points of projection from the concrete, since the strand-connecting eye-bars that were added and connected to the embedded bars at the time of spinning were allowed, during that operation, to follow the alignment of the wire strand. This latter condition also would have caused excessive stress in the eye-bars at the New York anchorage. The bending stresses in the projecting bars were practically eliminated by bolting the bars down by tie-rods and angle clips to a pair of I-beams erected beneath the bottom layer of eye-bars. The I-beams were rigidly connected to the falsework steel.

The change in alignment at the New Jersey anchorage caused greater difficulty, however, in the adjustment of the strands. It was necessary to

deflect the lower sets of strands and eye-bars to final position by pulling them downward at the strand shoes. The condition was a maximum at the lowest set of bars and decreased to zero at the middle set, above which point a gradually increasing upward deflection was required in the upper sets of bars.

The problem of adjusting and holding the lower sets of strands out of line was somewhat difficult. Studies were made as to the feasibility of placing anchor-bolts in the rock bottom of the eye-bar pits and also of excavating niches in opposite sides of the pits and inserting I-beams, or girders, to which the strands could be fastened. It was finally decided in the interest of economy and safety to use inverted truss arch ribs (see strand hold-down trusses, Fig. 21), hinged on the center line between cables and bearing against concrete pads built on the rock sides of the eye-bar pits. Four pairs of these ribs, located just below the heads of the eye-bars at the strand shoes, were used in each eye-bar pit. The six lower layers of strands were held in position with tie-rods connected between the arch ribs and the eye-bars.

In the finished cable the eye-bar heads are spaced by plate separators and the upward reactions of the lower layers of eye-bars are offset by the downward reactions of the upper layers.

The limited width of the eye-bar pits made it practically impossible to mount the strand shoes for spinning in the customary inclined plane above the eye-bar heads and still have room to spin four strands for each cable at one time. The strand leg, therefore, was designed to hold the strand shoe in a vertical plane in front of the eye-bar head. It consisted of two major parts: (1) A rectangular steel block of such dimensions as would fit between the pair of eye-bars with the upper end machined to bear against a pin placed through the eye-bar heads; and (2) a steel forging, the lower end of which fitted around the eye-bars in back of the heads, the upper end of which was provided with a swivel mounting for attaching the strand shoe. A removable side-plate bolted to the lower part of the forging prevented the eye-bars from spreading and slipping off the pin. Rectangular holes were cut through the forging and steel block for inserting a key which transmitted the pull of the strand to the block and, in turn, to the eye-bars. Pin-holes were placed in the ends of the key for attaching the adjusting tackle. Provisions were made for three different positions of the forging in relation to the block, corresponding to the computed variation in length of the strands. This was necessary to keep the strands as nearly as possible at a convenient height above the main-span footbridges.

Spinning Organization and Progress.—The erection of the cable-spinning equipment and the adjustment of the guide-wires was completed on October 18, 1929. Due to some minor adjustments required, actual spinning did not begin until October 21, when six trips of the spinning wheels were made on Cable *A* (the four cables were designated *A, B, C,* and *D,* from south to north) and three trips on Cable *C*. Spinning on Cable *B* and on Cable *D* was begun October 29 and November 11, respectively. The first set of four strands, on Cable *C,* was completed November 11.

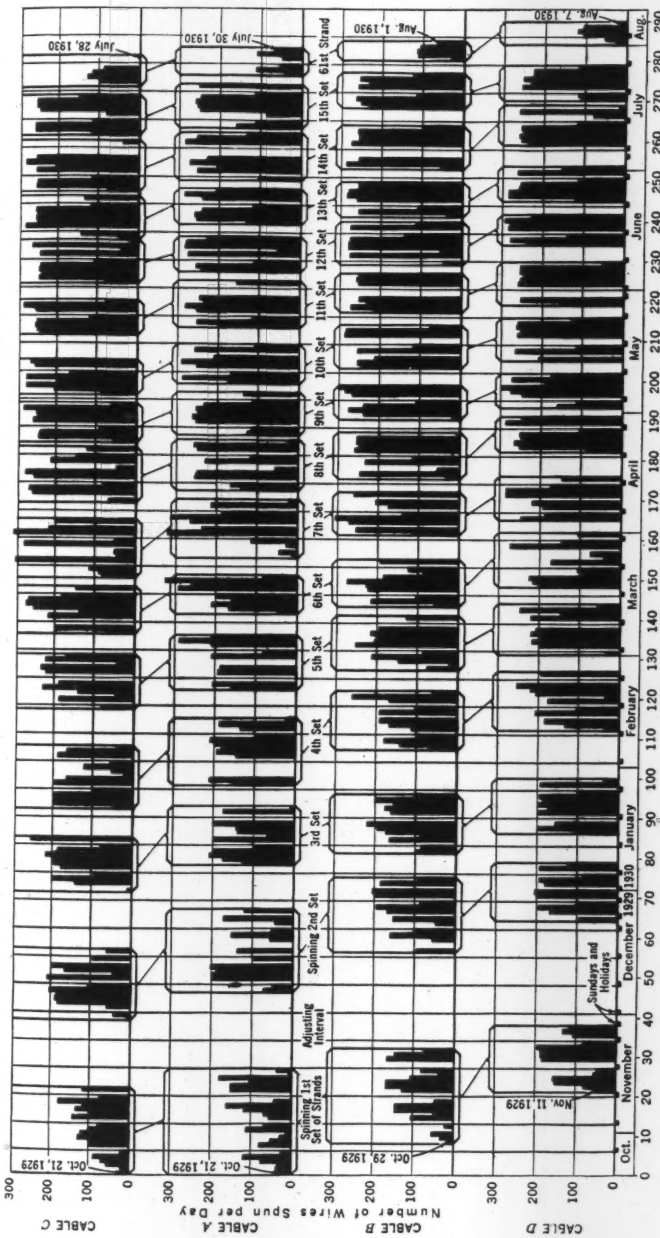

FIG. 28.—SPINNING CHART.

The accompanying chart (Fig. 28), records the daily progress in spinning the wire for each of the four cables. The rapid increase in efficiency of operation is clearly indicated. The first set of four strands required an average of 22½ calendar days in spinning and 21½ calendar days in adjusting and setting up for the next set of strands, as compared to the later average of 10 days in spinning and 2 to 3 days in adjusting.

At first, the workmen were organized in four spinning crews, each of which adjusted the strands when they had been completed. This arrangement did not prove entirely satisfactory, principally due to the inexperience of the men. Too much time was lost in adjusting a set of strands and in setting up for the next set. Accordingly, late in January, 1930, the men were re-organized into three spinning crews and one strand-adjusting crew. The three spinning crews shifted in cycles from cable to cable. The strand-adjusting crew worked in rotation on all four cables. Progress was carefully supervised and regulated, the crews being ordered to work overtime if necessary to avoid any delay to another crew. A decided improvement in progress resulted from this change.

Spinning was completed on Cable *C* on July 28, 1930, followed by Cable *A* on July 30. The remaining cables, *B* and *D*, were completed August 1 and August 7, respectively.

The maximum quantity of wire spun in any one day was on May 7, 1930, when 220 trips of the spinning wheels were made, placing 880 wires totaling about 236 tons for three cables. On this day, two of the spinning gangs worked 11 hours, and the third, 10 hours. The maximum wire spun in one cable in one day was on March 19, when 81 trips of the wheels were made in 12 hours on Cable *A*, placing 324 wires, or about 87 tons of wire. The maximum rate of spinning wire in one cable for a period of 8 hours, or more, was 7.34 trips of the wheels per hour, or about 8 tons of wire per hour.

Squeezing the Cables.—Immediately after the sixty-first strand was adjusted, the cables were squeezed in the main and side spans to within about 20 ft of the saddles. The squeezing operation broke up the individual strands, filled the voids, and formed a compact circular cable, as shown in Fig. 15. This was done in two operations, a preliminary squeezing—which made the cable partly round by forcing the corner strands in—and the final squeezing.

The preliminary "squeezer" consisted of a hexagonal frame that surrounded the cable with hydraulic-jacks mounted radially at the corners (see Figs. 29 and 30). Each jack was fitted with a shoe that pressed against the cable, the inside face being shaped to fit approximately over the corner strand and the one adjacent on either side. The squeezer was flexibly suspended from a carriage which also held the hand-pumps for the hydraulic-jacks. This carriage operated directly on the cable, each end being supported by a two-wheeled truck that could be lowered or raised to adjust its position. The wheels were made of wood and were shaped to fit the top layer of strands. Near the towers the carriages were propelled with hoisting engines placed at the anchorages and on the flatter portions of the cables with hand falls. Squeezing in the main span began at the towers. A single

FIG. 29.—PRELIMINARY SQUEEZER.

FIG. 30.—PRELIMINARY SQUEEZER; MECHANISM FOR APPLYING SEIZING BANDS.

application was made at approximately 100-ft intervals, working toward the middle of the span. On the return trip to the tower, applications were made at approximately 12-ft intervals. A 2-in. steel seizing band was placed around the cable at each application. This was tightened with an auxiliary jack and fastened with a wedge lock set in a cast saddle as shown in Fig. 30. The same operations were followed in the side spans, working from the towers to the anchorages and return. The main-span cables had to be squeezed principally at night when the strands were more nearly at uniform temperature. The hydraulic-jacks were 3 in. in diameter and were operated with oil pressures up to about 5 000 lb per sq in. The pair of jacks on the horizontal axis was connected to one pump, the upper pair to a second, and the lower pair to a third pump. At each application the routine was first to apply the horizontal jacks and then the upper and lower jacks together.

Two machines were used for the preliminary squeezing beginning July 31 and finishing on August 14. During this time, the spinning equipment at each anchorage was dismantled, and the hauling rope bents were removed from the footbridges.

The final squeezers were very similar to the preliminary squeezers. Each consisted of a circular frame surrounding the cable with twelve hydraulic-jacks mounted radially, as shown in Fig. 31. The jacks were $5\frac{3}{8}$ in. in diameter and were operated with oil pressures up to about 6 000 lb per sq in. The inside faces of the jack shoes were concave and, with the jacks extended, formed a circle 36 in. in diameter. The squeezers were supported by carriages operating directly on the main cables. These carriages, two for each side of the river, spanned a pair of cables, being supported by two double-wheeled trucks running on each cable. The wheels were made of wood and were shaped to fit the compacted cable. An electric pump was mounted on the carriage for operating the squeezers, which were hung on out-riggers extending from one end of the carriage. Hand-operated pulling jacks were used to connect the squeezers to the out-riggers to permit adjusting their position in relation to the cables. The carriages were propelled with the hauling ropes, two of the drive machines for this purpose having been transferred to the New York anchorage. These ropes were also cut and passed around idler sheaves attached to the cables at the center of the main span.

The carriages were designed to pass over the tower saddles into the side spans, but due to their weight, it was felt inadvisable to attempt this transfer. Equipment for handling the squeezers in the side spans was improvised in the field. The carriages were also intended for use in transporting and erecting the cable bands and suspenders, but this also proved impractical because the wheels cut the seizing bands unless they were protected with curved steel plates.

The final squeezing was begun at the middle of the main spar and progressed toward the towers. Applications were made at intervals from 2 to 4 ft, depending upon the resulting diameter of the cable. At first, all the

jacks were connected as a unit, but this did not give satisfactory results, because the final cross-section of the cable was found to be elliptical with the horizontal axis a maximum. The connections were then re-arranged so that the three jacks on either side worked together as one unit and the three on the top and bottom as separate units. This gave better results, but even then, the horizontal diameter of the cable varied from 1 in. to 2 in. more than the vertical diameter. It was felt that this variation would have no detrimental effect so the squeezing proceeded on this basis.

In the side spans the final squeezing was begun at the anchorages and progressed toward the tower. The electric pump was placed on a timber platform which rested directly on the main cables as shown in Fig. 32. The squeezers were lashed to the lower end of the platform, and were skidded up the cables with the platform by means of the hoisting engines at the anchorages.

The final squeezing began on August 19 and was completed on September 20, 1930. Concurrently with this operation, the cable bands were erected behind the squeezers, and hangers were installed for supporting the footbridges from the main cables as illustrated by Fig. 15. The storm system was also dismantled and the suspender cutting and socketing equipment set up on the top of each tower.

On completion of the squeezing operation, a survey was made to determine the actual positions of the cables in relation to the theoretical. This survey was made between the hours of midnight and sunrise to take advantage of the most favorable temperature conditions. Thermometers were placed on the cables at various points, selected to record the most probable average temperature. It must be recognized, however, that in cables of this magnitude, the actual temperature throughout the cross-section is not uniform and is, therefore, quite indeterminate. The results of the survey indicate a variation in main-span sags of 0.06 ft above, to 0.19 ft below, the theoretical. No attempt was made to check the cable sags in the side spans since, by necessity, they could not vary any appreciable amount, considering the small variation in main-span sags.

Erection of Cable Bands.—The castings of the cable bands for the main span were skidded out on the footbridges on timber sleds. The two castings forming a complete band were hoisted into position with chain blocks attached to a wood bent erected on the cables. The cable bands for the side spans were transported from the anchorages by carriages running on a pair of cables and propelled by the hoisting engines. Four castings forming two cable bands (one for each cable) were carried on each trip. They were hung from rods inserted through two of the upper bolt holes with the two halves partly surrounding the cable, but separated for clearance. After being shifted to position, the two halves were pulled together, and clamped to the cable with the permanent bolts, permitting the removal of the supporting rods.

The bolts for the bands were drawn up to 100 000-lb tension. Motors and drums, formerly used for operating the "come-alongs," furnished the

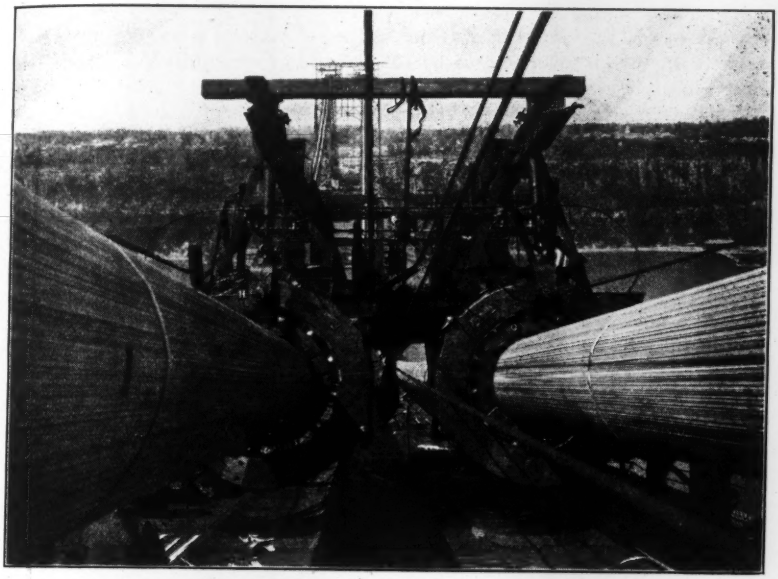

FIG. 31.—FINAL SQUEEZER IN MAIN SPAN.

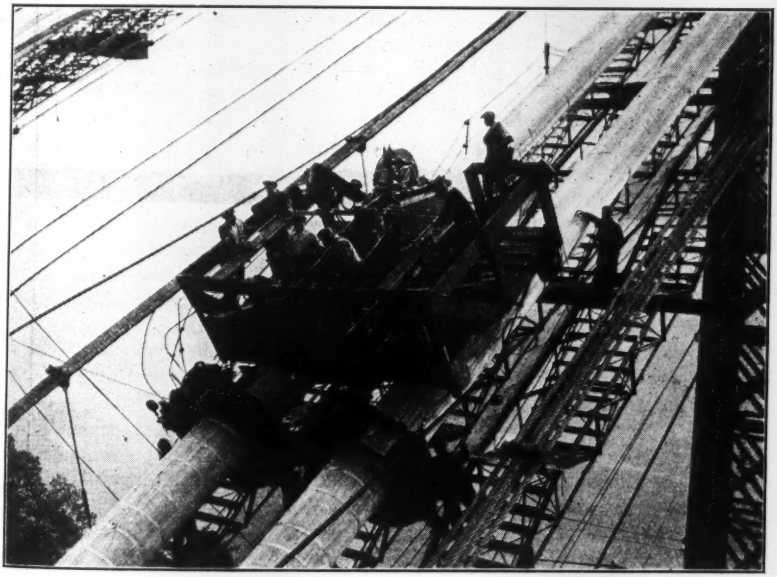

FIG. 32.—SQUEEZER IN SIDE SPAN.

power for pulling on socket wrenches for this purpose. This was done in two stages, a preliminary tightening that drew the bolts up to the approximate tension, and a final tightening prior to the erection of the floor steel, which was checked again just before the cables were wrapped.

The tension in the bolts was checked by strain-gauges. Dynamometers were also used to measure the pull on the wrenches and were found to give very dependable results. After the final tightening, the gaps between the two halves of the cable bands were caulked with lead wool to prevent moisture from entering the cables. The caulking was omitted in the bottom of several bands at the middle of the main span to provide drainage for any water that might work into the cables.

Releasing of Footbridge Ropes.—After the hangers for supporting the footbridges and cross-bridges from the main cables had been installed, the footbridge ropes were released from the decking. It will be recalled that the footbridge sections were clamped to the under side of the ropes, which thus facilitated this operation. The release of the ropes presented no problem in the side spans, because the difference in sag between the loaded and unloaded ropes was comparatively small. In the main span, however, the difference in sag between the loaded and unloaded ropes was about 10 ft. The footbridge panels were cut loose starting at the towers and progressing toward the center of the main span. Upon reaching the quarter-points, the operation was stopped and resumed at the center of the span. When the load on the ropes had been released so that the sag had decreased an appreciable amount, the panels were kept lashed to the ropes for several points behind the point of loosening and slacked off progressively to maintain a smooth transition between the panels hung from the main cables and those still attached to the ropes.

Suspenders.—The footbridge ropes were measured for suspender lengths while hanging in free suspension, those for the side spans and the main span being connected at the tower by short pennant ropes.

A careful schedule was prepared in advance showing the length and location of the suspenders to be cut. In computing the lengths, consideration was given to the variation in stress at different points along the ropes at the time of measurement. These stresses were based on a condition of balanced horizontal components at the towers assuming the rope to hang level with the bottom of the main cables at the center of the main span, a condition which was obtained by supporting the ropes at the towers with hangers attached to rollers. The lengths were also corrected to correspond to variation in cable sags. No allowances were made for later adjustment with the use of shims.

The measurements were made at night to take advantage of more uniform temperature conditions. This also permitted suspending and reeling in the ropes during the daylight hours. Two ropes were measured each night, the ends and mid-point for each suspender being located and marked, together with its panel point number. White stripes were also painted at intervals along the rope to indicate the lay at the time of measurement. This was done to avoid changes in length resulting from possible twisting

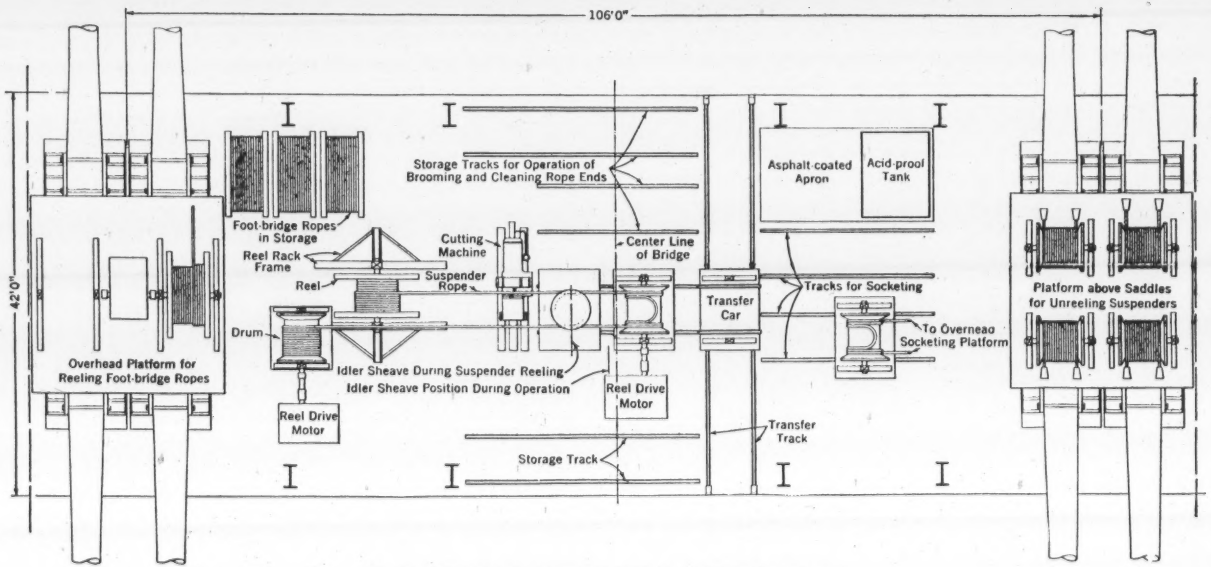

FIG. 33.—SUSPENDER CUTTING AND SOCKETING EQUIPMENT.

of the rope during handling subsequent to measuring. The next morning the ropes were lowered to rest on the footbridges and the main-span section was "burned" in two at a waste part of the rope, which was usually near the center of the main span. The main-span ropes were then reeled up at each tower on wooden reels mounted on the platforms over the cable saddles, followed by the side-span ropes.

Duplicate cutting and socketing equipment was placed on the tower tops. Due to the restricted area available, the suspenders had to be handled on reels (see Fig. 33). The one containing the half main-span length or side-span length of rope was mounted adjacent to the inside cable saddle with its axis parallel to the center line of the bridge. The end of the rope was taken from the reel and cut at the first end of a suspender with a hydraulic shear. The sheared end was then passed around a horizontal idler sheave back to a steel drum near the rope reel. The rope was reeled up on this drum until the mid-point of the suspender was half-way around the idler sheave. While in this position, two zinc buttons were cast on the rope at the correct distance on either side of the mid-point to serve as retainers for the clamp, which would hold the two parts of the suspender together under the main cable. The loop or bight of the suspender was then placed on a second drum mounted on a hand-car behind the idler sheave, and the two parts of the suspender were reeled up, pulling the rope from the reel and the adjacent drum. It was then sheared at the mark locating the other end of the suspender, which completed the cutting operation. Zinc seizing blocks with steel inserts fitting the strands of the rope were clamped on either side of the cutting points, which prevented the strands from untwisting and also served as stops for setting the sockets. After cutting, the drum was shifted to a storage track where the wires of the rope were broomed out and cleaned with acid in the customary manner.

The suspenders were socketed on an elevated enclosed platform, the ends of the rope being pulled up from the drum. These ends were clamped separately beneath the platform, to short sections of I-beams sliding between vertical guides and actuated with double-acting hydraulic-jacks. Each end was raised until the top of the seizing block was flush with the top of the guides. Two halves of a steel block supported by the guides were bolted together, with the rope passing through a tapered hole in the block. Jacking the rope down through this block forced the broomed wires together into a compact circular group. The downward jacking was stopped when the ends of the wire were about flush with the top of the block. A split collar, tapered on the bottom to fit down into the hole in the block, was then placed around the wires. The socket was placed on the top of the collar over a thin lip which fitted into the hole in the socket. After clamping the socket to the guides, the rope was jacked upward, forcing the ends of the wire into the socket. The collar was then removed and the rope was raised to bring the seizing block in contact with the socket. Melted zinc from electric furnaces was then poured into the sockets.

Day and night shifts were employed in order to maintain the schedule of two footbridge ropes per day. The suspenders for the north pair of cables

were cut from the ropes supporting the south footbridge followed by the suspenders for the south cables from the north footbridge ropes.

Erecting the Suspenders.—The drums containing the longer suspenders, which could not be erected by hand, were placed in racks on top of the tower saddles. The ends of the suspender were allowed to unreel and slide down the footbridges on either side of the cable on which the suspender was to be erected. At the designated panel point the ends were placed over wood-lined sheaves and lowered through the footbridge as shown in Fig. 34. Manual brakes retarded the unreeling of the suspender until the bight was free of the drum, after which a wire rope leading to the hoisting engine at the anchorage, and connected to a bridle clamped to the suspender below the zinc buttons, was used to control the movement of the suspender and lower it into position.

Wrapping the Cables.—The main cables are protected from the weather by a layer of galvanized wire wrapped around the cables between bands. At the bands, the wrapping terminates in recesses machined at the ends of the castings. Water-tight connections are made by caulking with lead wool.

The wire was applied under tension in such a manner as to insure close contact between the turns. A heavy coat of red lead paste was applied just in advance of the wrapping and one coat of red lead paint and two coats of aluminum paint were applied afterward. This method produces an effective seal around the cables and has long been adopted as being the most satisfactory way to protect parallel wire cables.

In order to prevent the wrapping wire from becoming loose due to lengthening of the cables, it is desirable to postpone the wrapping until all, or at least the major part, of the dead load of the bridge is in place. For this reason the wrapping operation did not start until July 9, 1931, after the concrete roadway and sidewalk slabs were in place. It was completed on October 1, 1931.

The wire was placed by two types of machine under a tension of 400 to 500 lb as nearly as could be estimated and maintained. One applied the wrapping at the rear end as it progressed, and the other applied it at the forward end; the first, a "pusher" type, operated directly on the cable ahead of the wrapping, and the second, a "puller" type, moved along on the wrapping. Except for the direction of rotation, the two machines were essentially identical and were adopted to eliminate all hand-wrapping, the intent being to wrap approximately one-half of each panel with a "pusher" machine and then complete the panel with the "puller" machine. In actual service, however, the latter did not prove satisfactory as there was a tendency to injure the wrapping while sliding over it. Furthermore, the "puller" machine interfered with the inspection of the freshly wrapped area since it had to advance approximately 5 ft before exposing the wrapping on top of the cable; thus, if any defects existed it was necessary to unwrap for a distance of 5 ft in order to correct them. Accordingly, the "pusher" machine was used wherever possible, the remaining part in each panel being wrapped by hand.

Some difficulty was experienced, mainly in the side spans, with loose wrapping over a flat strip on top of the cable. This difficulty was overcome

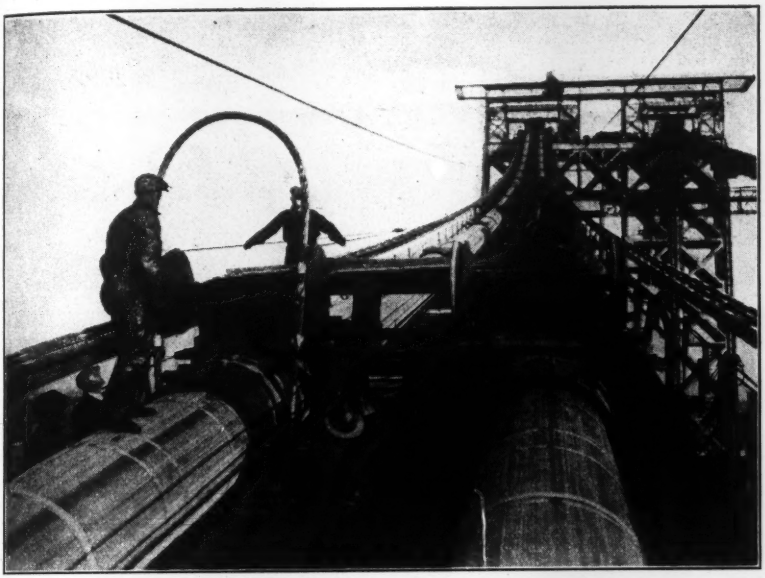

FIG. 34.—ERECTING SUSPENDERS.

FIG. 35.—WRAPPING MACHINE.

by filling out the flattened area with several layers of sheet lead. These sheets were of varying width and were beveled along the edges. A strip of galvanized iron was placed on top of the lead to keep the wrapping wire from pressing into it, which would have prevented the necessary crowding action between the turns of wire.

Four machines of each type were provided, one of which is shown in Fig. 35. They were assembled on sheet-steel saddles about 5 ft long. The saddles were shaped to bear directly on the cables, a cast-steel yoke being bolted to one end. From the yoke were hung two electric motors, one on either side, with their shafts slightly above the horizontal axis of the machine. Reversible motors of 10-hp capacity, capable of driving the machine at 50 rpm, were first used, but later were replaced with 15-hp motors rotating at the same speed. An auxiliary motor operating through a "Bendix" unit and reduction gears to one of the main motors supplied a slow-speed drive for starting or wrapping the cable adjacent to the band. The motors were geared directly to a ring gear surrounding the cable at about the center of the saddle. The ring gear ran in a groove machined in a bronze bearing bolted to the saddle. The wrapping wire was carried on two reels on either side of the ring gear. These reels also ran in grooves in bronze bearings bolted to the saddle.

Band-brakes attached to the gear and rubbing on the adjacent flanges of the reels served to rotate the reels for loading with wire and also to produce the tension in the wire for wrapping. These brakes were adjustable for regulating the wire tension. They could also be released by a lever device. Supplementary brake-shoes, attached to the saddle, could be tightened to bear against the reels for unwrapping. A flyer ring, mounted in a bronze bearing at the other end of the saddle, was connected to the ring gear by two notched plates bolted to the ring and fitting over lugs cast on the side of the gear. These had to be removed each time the reels were loaded with wire. Four fingers were evenly spaced on the outside face of the flyer ring. These were made in the form of radial plungers sliding in housing bolted to the ring. The plungers were fitted with steel shoes that bore against the last few turns of wrapping wire. Lips, extending down from one edge of the shoes and just clearing the cable wire, produced a crowding action that forced each turn of wire in contact with the previous turn. Coiled springs with screw adjustment bearing against the plungers, kept the shoes in contact with the wrapping wire.

The wires were led from the reels on opposite sides of the machine passing over brass guides and through holes in the gear and flyer ring to the cable. The overturning effect on the machine due to the tension in the wires was resisted by pipe outriggers bearing on the adjacent cable. The reels, gear, and flyer ring were made in two sections, bolted together. They were separated each time the machine was shifted to a new panel.

Wrapping started at the center of the main span and at the anchorages and progressed toward the towers. No automatic mechanism was provided for advancing the machines along the cable as it was felt that the crowding action of the shoes bearing against the wrapping wire would be sufficient

to move the machine ahead. It proved necessary, however, to provide some auxiliary means to accomplish this. Experiments were made with various systems of counterweights, but the method that proved most satisfactory was simply to pull the machine ahead with a hand-winch fastened to the cable from one panel to two panels in advance of the wrapping. Manila-rope snubbing lines working against the winches were used to prevent the machines from moving in jumps.

The wrapping wire was shipped to the site on the reels used for the cable wire, which were fitted with axles and were mounted in racks on the roadway at the center of the main span and adjacent to the towers. The wire was led from the reels up to the footbridges and thence along the footbridges to the point of wrapping, those on the wrapping machines being filled from the most convenient source. The reels were rotated in the same direction while being filled as the machine rotated in placing the wire. Portable, electric, butt-welding machines were used to splice the wire.

Floor Steel Erection

General Procedure.—On September 12, 1930, while the contractor for the cables was engaged in final cable compacting operations, the steel contractor began to erect the floor steel of the towers. A Chicago boom was assembled on the lower portal of each tower, and the floor steel within the tower was erected by this means. The travelers for erecting the floor of the main span were then assembled at the towers. When the suspender ropes were in place these travelers were ready to go forward with the erection of the steelwork of the main span, proceeding from the towers toward the center of the river, hoisting the steel directly from car-floats anchored in the river, and moving ahead as each panel was completed.

The side-span travelers were assembled at the towers after the main-span travelers had advanced a few panels. Because of the location of the side spans over land, the materials for these sections were hoisted from car-floats to the floor level on the main-span sides of the towers, placed on hand-cars, and pushed over the previously erected side-span steelwork to a point from which the material could be picked and placed in position by the side-span traveler.

The Main-Span Travelers.—Two **A**-frame derricks, with cable back-stays, placed side by side, made up each main-span traveler. The derricks of the New Jersey traveler (shown in Fig. 36) had been used by the contractor to erect the Cooper River Bridge, at Charleston, S. C., and those of the New York traveler had been used in erecting the Outerbridge Crossing over the Arthur Kill to Staten Island, New York, and the Mt. Hope Bridge, in Rhode Island. They were entirely different, therefore, as to detail.

The derricks in Fig. 36 were link-connected together at their sills, but were otherwise entirely independent of each other. The **A**-frames had a comparatively short spread at the sill (18 ft $5\frac{1}{4}$ in.) and, therefore, to improve stability during erection they were braced further by side guy legs of 15-in. channel sections on the north side of the north derrick and on the south side of the south derrick. The derricks were equipped with 100-ft booms.

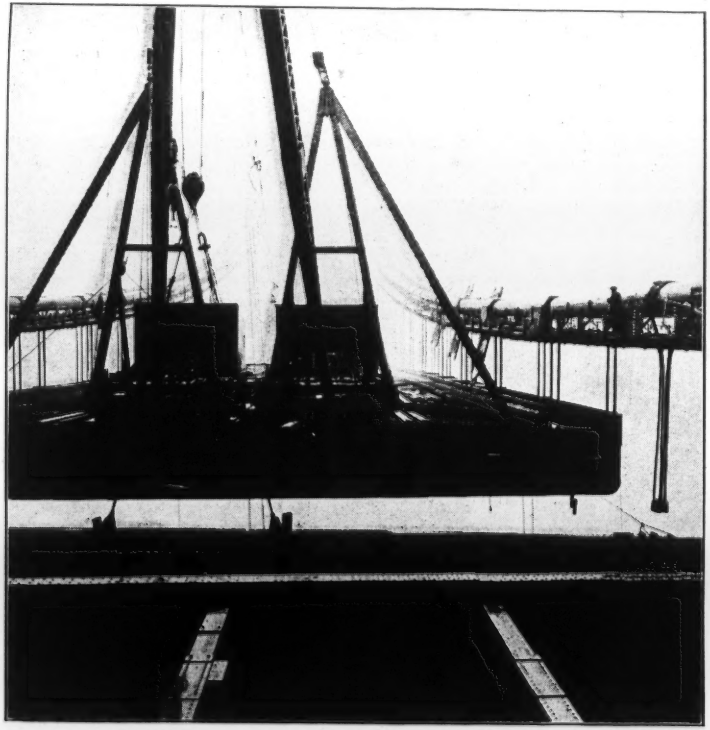

FIG. 36.—MAIN-SPAN TRAVELER, NEW JERSEY SIDE.

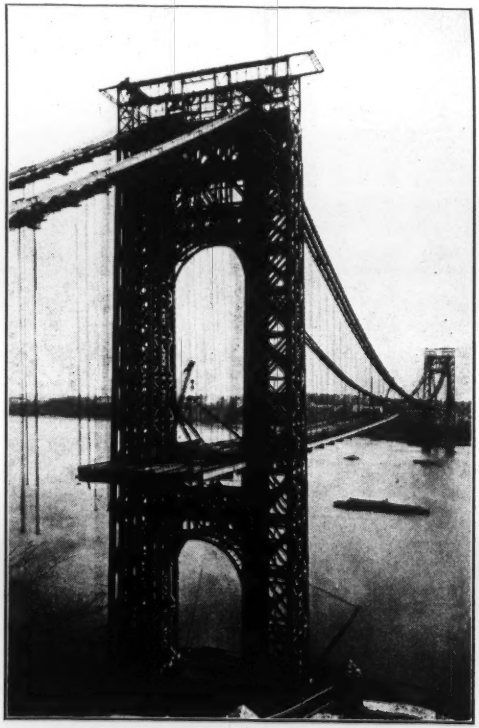

FIG. 37.—VIEW SHOWING FLOOR ERECTION.

They were mounted on trucks that traveled on rails fillet-welded to 10-in., Bethlehem girder sections laid on the transverse beams. In position for erection, the derricks were raised from the trucks, the loads being transferred to the floor-beam by means of an assembly of screw-jacks and castings under each leg. At the rear end, the derrick was lifted from its trucks by 12 by 12-in. blocking wedged between the frame and the steelwork and, at the same time, tied down by rope slings passed under the transverse I-beams.

Two gasoline engines (three-drum, three-speed, 150-hp), operated the New Jersey derricks. The booms were swung by block-and-fall swing lines operated from the engines, and the derricks were advanced from position to position by means of manila rope falls. The entire traveler with the engines weighed approximately 233 tons.

The traveler derricks on the New York end had a greater spread of the A-frame legs at the sills—about 23 ft. They were link-connected at the sills, as was the case with the New Jersey derricks. The A-frames were connected at the tops by two parts of 1-in. cable, and were guyed sidewise by four parts of 1-in. cable made fast to the floor-beam north and south of the traveler during hoisting operations. These guy lines replaced the struts used for the New Jersey derricks. They were mounted on trucks on rails, but were raised from the trucks by means of wedge-jacks and blocks at the sills, and blocks with wedges at the rear end of the frame before raising the loads.

Each derrick was equipped with an 85-ft boom and powered by a 4-drum, 150-hp, electric engine. These 4-drum units were those which, when coupled with units of three additional drums, had composed the seven-drum engines used in erecting the towers, as previously described. The four drums operated the main load, the auxiliary load, the boom falls, and the falls for advancing the traveler. The booms were swung by block-and-fall swing lines and a small electric motor on the traveler. The weight of the New York traveler with the engines was approximately 281 tons.

The use of electric engines for one traveler and gasoline engines for the other is explained by the fact that, although gasoline engines were preferable, only the pair for the operation of one traveler was owned by the contractor, and the purchase of additional units was not considered to be justified since the electric engines were already available at the site. The electric engines required the use of heavy conductor cables, however, which were costly, and were expensive to handle in the field. The choice of the New York traveler for the use of the electric engines was dictated by the cheaper power rate on that side of the river.

Structural Steel in the Main Span.—As was the case in handling the steelwork for the towers, all the steel for the floor system was shipped from the shops to ground storage at the New Jersey terminal yards of the railroads, where the pieces were reloaded on cars in a manner that made possible their removal in the sequence required in erection. Except for the floor steel of the first panel at the New York tower, all members were hoisted from car-floats moored to barges in the river, directly below the panel being

erected. The barges were anchored in the river by lines at the four corners. These lines permitted the barges to shift a certain amount without changing the location of the anchors. When required, the anchors were changed to new positions by tugs.

The first steelwork of the suspended structure was erected in the main span at the New Jersey tower on October 28, 1930. According to the usual procedure, as followed through in erecting a typical panel of floor steel, the two booms of the traveler together raised the floor-beam, weighing approximately 62 tons. During this operation, the weight was equally distributed between the derricks by the use of a balance sheave.

While on the car-float, light scaffolds were attached near the ends of the floor-beam to give the bridgemen positions from which to work while entering the socketed ends of the suspender ropes in the seat connections provided on the floor-beam. Workmen did not ride the floor-beam during the hoisting operation, but walked out to it on 12 by 12-in. timber struts, used to space the floor-beam from the panel previously erected.

The fastening of the eight suspender-rope sockets to each end of the floor-beam was the most tedious part of the operation. Reference has been made to the lines painted on the suspender ropes during the process of measuring, the object being to permit attachment of the rope without excessive twisting or untwisting. For the longer ropes, a single turn variation from the condition under which the rope was measured may so alter the lay as to change the length by as much as $\frac{1}{2}$ in. It was highly desirable, of course, to maintain the measured length of suspender. It was intended by observation of the painted line to make sure that no twists occurred. In actual practice it was found to be exceedingly difficult to determine whether or not the long suspenders were entirely free from twists. The line could be followed for a considerable distance, but under certain conditions of light, especially in hazy weather, it was almost impossible to follow the painted line to the top of the suspender.

After the floor-beam had been set in place, each of the two derricks proceeded to operate independently in filling in the remainder of the steel required in the panel. The first members were the wind chords, which were entered in the floor-beams from below. These were followed by the stringers, bracing, and transverse beams. The bulb-beams were hoisted to the deck, but were not placed in position by the travelers. Otherwise, the entire floor steel was erected complete in one pass. Because of the relatively great mass of the cables, such a procedure was possible without producing objectionable distortions in spite of the great weight of the travelers. Saddle motions, cable bending, saddle friction, operating grades for travelers, and general as well as local floor distortions for this condition were carefully calculated in advance to be within safe and practicable limits.

The floor steel followed a curve that was concave upward until erection progressed to approximately Panel Point 38. (See Figs. 37 and 38.) At this point, the curve became flat and thereafter it was concave downward. The curvatures were much less pronounced than in other relatively lighter structures. The deformations of the main floor during the various stages of

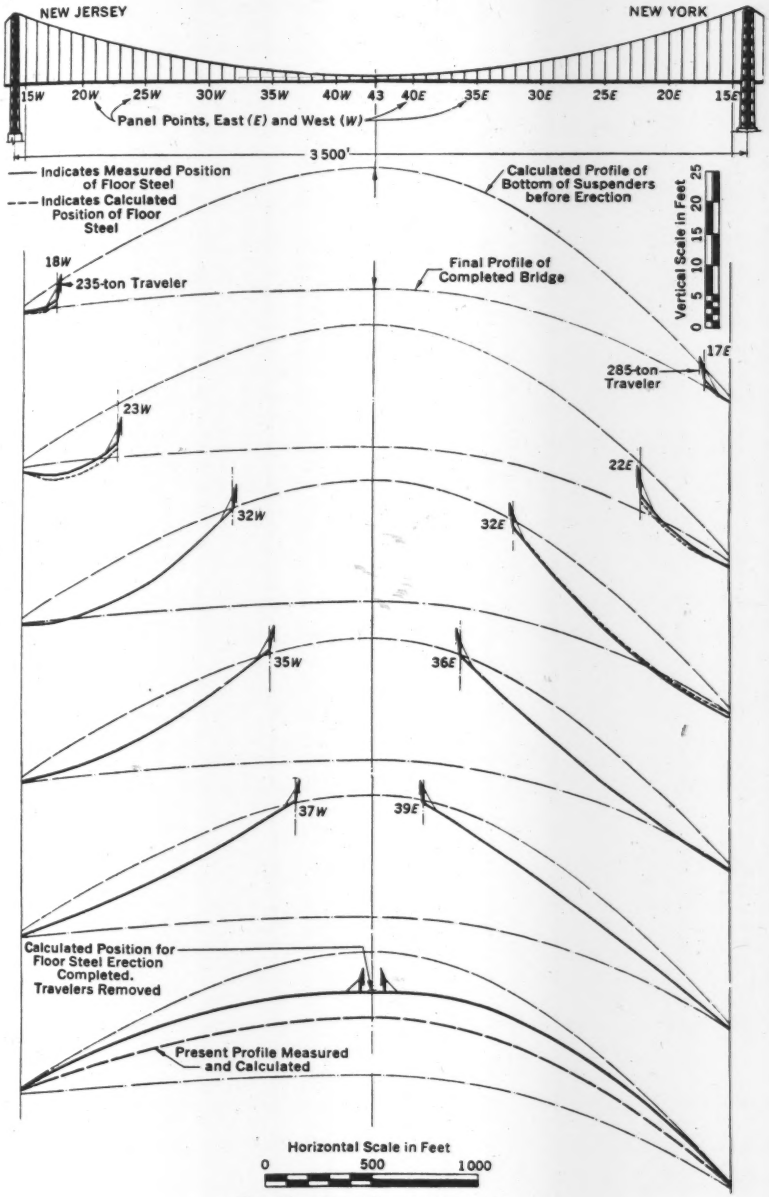

FIG. 38.—MAIN-SPAN PROFILES.

construction are shown on Fig. 38, for a dead floor load of 8 000 lb per ft of bridge. For purposes of comparison all data were corrected to a normal temperature of + 50° F. In a few cases the corresponding theoretical profiles are plotted.

However, some difficulties were experienced because of the distortion, principally with the wind chords and the diagonal wind-bracing. The ends of the chord members are milled at right angles although the final curvature of the chords is concave downward. The secondary stresses induced by this curvature were calculated as being too small to warrant milling at the slight angle required.

As erected, the abutting chord members were in contact at the top and open at the bottom of the joint. It was impossible, therefore, to fit up the splices completely, immediately after erection; and yet, because of the action of the wind on the erected part of the floor system, it was essential that a certain minimum number of bolts be placed in order to carry the stresses safely. The desired results were obtained by fitting up the splices in the cover-plates, the remaining splice-plates being left loose. As the erection progressed, a careful check was made from time to time to determine the profile, and whenever the chords over two or three panels tended to approach a straight line, the remainder of the splice at these points was fitted up. No drifting was permitted when it would be injurious to the holes. Special drift-pins of exact diameter were driven by air hammer.

The floor-beams are framed to hang vertically in the completed bridge, rather than normal to the roadway. As erected, the chord members sloped upward away from the traveler, causing a considerable angle between chord and floor-beam connections. Therefore, when first entered from below, the chord was held up to the forward floor-beam by bolts through the cover-plate. The tilting of the floor-beam necessary to complete the connection occurred when the stringers were erected. These stringers were bolted only through the holes in the seat connections at both ends as first erected. The eccentric load from these stringers on the forward floor-beam also tipped it sufficiently to permit "making" the top holes of the stringer connections. Theoretically, the fact that the chords hinged at the rear splice-plates, 8 ft off the panel point, would cause the chords and stringers to come into the floor-beam at slightly different angles. However, the difference was not noticeable in actual erection.

Some difficulty was experienced in erecting the diagonal members of the wind-truss system. These members lie below the plane of the stringers. Accordingly, during the time that the profile of the floor followed a curve concave upward, the actual distance between the points of attachment of the ends exceeded the final amount. At times, it appeared to be almost impossible to make the end connections of both the diagonals in a panel. However, by the use of small bolts the connections were made, although the final fitting was of necessity deferred until a more favorable profile was obtained.

During the early stages temporary connections of the chords to the towers were required to resist the lateral deflection of the floor steel due to wind. Calculations indicated that, with the steel erected seven panels out from the tower, the lateral deflection at the last panel point erected could be as much as 13½ ft under moderately heavy wind pressure. This deflection would cause a longitudinal motion of 45 in. in one chord, assuming the other chord to be fixed. Since this amount would exceed the maximum allowable, it was necessary to bolt the end chord members to temporary connections at the towers.

As erection continued beyond the seventh panel from the tower, the computed lateral deflection and consequent potential motion at the tower became less, while the wind stresses naturally became greater. It was necessary, therefore, to release the temporary connection shortly thereafter although blocking between the chord and tower was left in place until the traveler reached the twenty-second panel from the tower.

When the erection of the floor steel was begun, the cable saddles on the towers were set approximately 23 in. shoreward of their ultimate position, in order to allow for the lengthening that would take place in the side-span cables due to the added dead load of the completed structure. Cable bands and suspenders at the first panel point adjacent to the towers in the main span were thus displaced horizontally so that the space between the floor-beam—when hung from the suspenders—and the tower was 23 in. less than normal. With normal shims of 6 in. at the bumper strut a latitude of 10 in. was available. It was thus necessary to push out the floor-beam at least 13 in. in order to erect the chords and stringers in the first panel.

The amount of shoreward horizontal displacement of bands and suspenders at succeeding panel points, and the corresponding "push out" of the floor-beam required to make connections, decreased progressively, the displacement becoming zero at the eighth panel point from the tower. Beyond that point the displacement gradually increased in a riverward direction attaining a maximum of 8 in. at the seventeenth panel point from the tower and then gradually decreasing to zero at the center of the span, twenty-eight panels from the tower.

Closing Operations—Main Span.—The cable bands are spaced along the cables in such a manner as to bring the suspender ropes to a vertical position at 60-ft horizontal spaces when both decks are in place. With only the steelwork of the upper deck in place the average location of the cable band was such that if both half-span units were cut free at the towers and allowed to float into equilibrium position the gap between the last two panel points at closing would be 1¼ in. shorter than the closing chord members under normal temperature conditions, this being due to the inclination of the suspenders throughout the central part of the span where they are closer together at the cable than they are at the floor.

Computations indicated that a 26-ton jacking force, if applied at mid-span, would be required to hold the two units apart the distance necessary for inserting the closing chord. Measurements were taken of the gap between

milled faces of chord splices when steel had been erected to within three panels of closing. The New Jersey half-span unit of floor steel was then pulled toward the tower by means of turnbuckles, an amount necessary to cause an opening sufficient to allow for the length of steel to be inserted, plus the shortening of the gap due to the deflection of the two half-span units while adding the last three panels, plus an allowance for a possible change due to temperature rise subsequent to the measurement, plus a nominal $\frac{1}{2}$-in. allowance for insertion of the closing chord.

The shortening of the gap due to the profile changes subsequent to the measurement was computed to be 1 in.; and the effect of temperature would close the gap $2\frac{1}{2}$ in. for each rise of 10° in temperature.

As it developed the temperature rise was slightly more than anticipated although not enough to cause any difficulty in closure. The closing chord members were set in place with $\frac{1}{8}$-in. clearance. The last panel was erected at the center of the main span on December 29, 1930, 62 days after the erection of the first suspended steelwork (October 28, 1930). Fifty-eight panels of steelwork were erected by the main-span travelers in 40 working days, an average only slightly less than $1\frac{1}{2}$ panels per day.

The Side-Span Travelers.—The travelers for the erection of the side spans were alike. Each consisted of one of the stiff-leg derricks used on the tower traveler. Because of the comparative shortness of the cables in the side span little change in cable deflection was to be considered in erecting the steel, and for the same reason, progress in erection could be allowed to lag far behind that of the main span. Of course, the reverse would not be true; side-span steel erection could not precede the main-span steel erection. The single-boom stiff-leg derrick, while slower, was, therefore, entirely satisfactory; in fact, it was the most satisfactory type of equipment for swinging back to handle members trucked to it over the floor steel for erection in the following panel.

The derricks had 85-ft booms and were designed to have a capacity of 84 tons at 60-ft radius and 65 tons at 80-ft radius. The heaviest piece to be handled was the 62-ton floor-beam at a 60-ft. radius.

It will be recalled that the engines for the operation of the tower erection travelers were near the base of the tower on the shore side. One of the three-drum units (an electric engine of 150-hp capacity) left in its original position, was used to operate the side-span traveler. The "cat-heads," which had been used in hoisting the tower erection traveler were located at convenient points to carry the leads from the derricks to the engine. Special sheaves were also provided to support the leads above the floor steel.

Structural Steel in the Side Spans.—The steel for delivery to the New Jersey side span was brought from ground storage in car-floats which were moored on the river side of the tower foundation piers. The remaining stiff-leg derrick of the tower traveler was set up within the portal and on the south side of the roadway at the river face of the tower, one leg being replaced by a short connection to the tower steel. In this position, the boom reached over the side of the floor steel on the main span and hoisted members to the deck level. There was just sufficient space to manipulate the

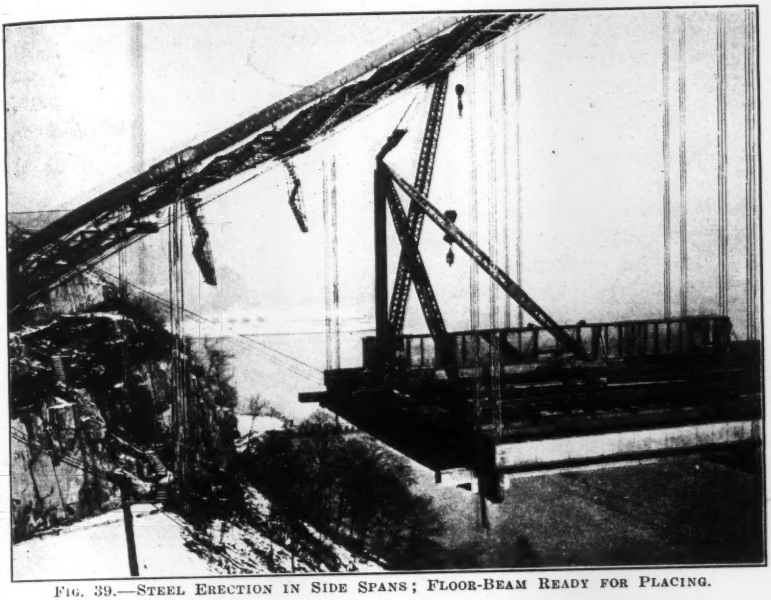

FIG. 39.—STEEL ERECTION IN SIDE SPANS; FLOOR-BEAM READY FOR PLACING.

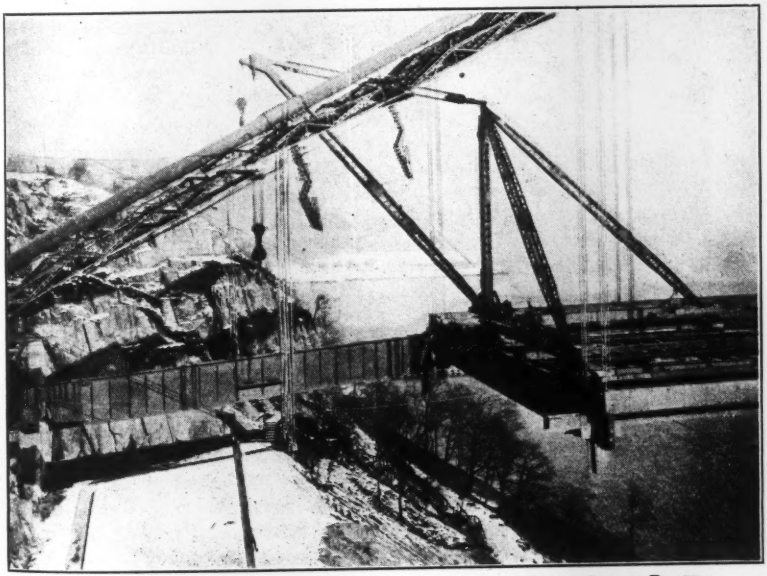

FIG. 40.—STEEL ERECTION IN SIDE SPANS; FLOOR-BEAM BEING SET IN PLACE.

main floor-beams between the face of the tower and the first group of suspender rope. The members were placed on trucks, operating on the bulb-beams as rails, near the north side of the roadway, and were then carried to a position in the last panel erected, within reach of the traveler boom. (See Figs. 39 and 40.)

The arrangement for hoisting the steel to the roadway level on the New York end was somewhat different. It will be recalled that the New York tower foundations are back a short distance from the water's edge, necessitating a temporary mooring wharf for the car-floats. Accordingly, the stiff-leg derrick for hoisting side-span materials was skidded to position in the second panel of the main span, from which position it could reach over the side of the floor steel and hoist the steel members to the floor level for trucking to the traveler.

The floor steel was erected in the same order as was followed on the main span, although somewhat different conditions were to be met. The side-span travelers practically always worked on a down-grade ahead. As in the main span, the floor-beams were framed to hang vertically in the completed bridge and, therefore, required tipping forward to permit matching holes in the connections between the chords and the floor-beams.

At the beginning of erection the cable bands and suspenders at the first panel adjacent to the tower in the side span were approximately 23 in. shoreward of their normal position because of the setback of the tower saddles as previously explained. The displacement of cable bands and suspenders shoreward was, of course, progressively less at each panel point toward the anchorage until the last was only 3 in. shoreward horizontally. All these conditions were anticipated and met by pulling the floor-beams back toward the traveler to make the chord connections, and it was necessary to lash the chords to the tower to avoid pulling the first panel stringers off their seats on the towers.

As in the main span, temporary connection with blocking was used during the erection of the first few panels of floor steel to resist the longitudinal motion due to lateral deflection of the floor steel under wind pressure.

Erection operations in the side spans were slower than those of the main span, two or three days being required for each panel. The New Jersey side was completed on January 8, 1931, and the New York side on January 21, 1931.

Tower Deflections, and Positions of the Cable Saddles.—Variations in position of the tops of the towers for various conditions of loading are shown in Table 2. Typical average conditions, which include the maximum range during cable spinning and erection of the floor steel are shown. For simplicity, the motions are given for the New Jersey tower only. Except for slight discrepancies that are unexplainable, these motions agree reasonably well with values calculated in advance.

The effect of the footbridge ropes as erected, but not adjusted, was to cause a slight riverward deflection. This was caused by the fact that tensions in side-span and main-span ropes were approximately equal at the time, resulting in an unbalanced riverward H-component. Considerable

TABLE 2 — DEFLECTIONS OF NEW JERSEY TOWER DURING CABLE SPINNING AND ERECTION OF FLOOR STEEL

Date	Temperature, in degrees Fahrenheit (Normal, 50° F.)	Average deflection from vertical toward the river, in feet	Average position of cable saddles relative to the top of the tower (toward the shore), in feet	Remarks
July 1, 1929.....	70	0.00	1.89	Normal unloaded tower.
August 8, 1929.....	84	0.03	1.89	Footbridge ropes erected, but not adjusted.
September 11, 1929..	70	0.16*	1.89	Footbridge ropes adjusted.
October 22, 1929....	70	0.47	1.89	Footbridges completed.
November 27, 1929..	38	0.56	1.89	†First set of strands in permanent position for Cable *C* and spinning position for other cables.
December 12, 1929..	26	0.51	2.00	First set of strands in Cables *A*, *B*, and *C* adjusted; partly adjusted in Cable *D*.
September 30, 1930..	60	0.32	2.00	Cables completed and footbridge ropes released.
November 13, 1930.	60	0.39	2.00	Four panels of main-span steel erected adjacent to each tower. No side-span steel erected.
November 18, 1930..	54	0.10	1.65	Five panels of main-span steel erected adjacent to each tower.
December 11, 1930..	42	0.25	1.65	Eighteen panels adjacent to New Jersey tower and sixteen panels adjacent to New York tower in main span; three panels in New Jersey side span.
December 11, 1930..	42	0.06	1.40	Eighteen panels adjacent to New Jersey tower and sixteen panels adjacent to New York tower in main span; three panels in New Jersey side span.
May 13, 1931.....	50	0.36	1.40	Floor steel completed.

* Deflection toward the shore.
† The four cables were designated *A*, *B*, *C*, and *D*, from south to north.

tightening was required for adjustment in the side-span ropes, which pulled the top of the tower shoreward. The original intention was that the towers would be vertical for this condition at normal temperature. It will be observed that the greatest deflection occurred with the first few completed strands in their spinning positions, the tower being gradually drawn more nearly vertical by the successive adjustment of completed strands. Furthermore, the tendency of the cables to cause excessive riverward deflections during the erection of the floor steel was offset by adjustment in the position of the cable saddles. The changes in position were made by jacking, the position being maintained by temporary blocks inserted between the river side of the saddles and the steelwork of the tower.

When the structure had been completed to its present stage, the saddles were jacked to a position $7\frac{1}{8}$ in. shoreward of the center line of the top of the tower for the New Jersey tower and $5\frac{13}{16}$ in. shoreward for the New York tower, these positions being such that the towers would assume vertical positions for normal temperature under the dead load of the completed upper deck. The saddles are maintained in these positions by permanent steel blocking.

Construction Schedule

A definite construction program, with dates to be met, was established in the contracts. Certain modifications were made in the original schedule at the time the cable contractor changed from his original plan of successive erection to that of simultaneous erection.

According to the schedule as revised, the erection of the steelwork of the two towers was to be completed by June 1, 1929, to such an extent as to permit the erection of grillages, saddles, and cable falsework, and all work on the towers was to be completed by September 1, 1929. Cable construction, except wrapping, was to be completed by November 1, 1930, in order that erection of the floor system might begin on that date. and the floor system was to be completed by May 1, 1931, to such an extent as to permit a start to be made on the construction of the concrete deck. All work on the steel floor system was to be completed by July 31, 1931.

The dates of the construction program are cited in order to indicate how closely the advance program was adhered to on work of such magnitude. Assembly of erection equipment was begun on the foundation of the New Jersey tower on May 1, 1928, and on the New York foundation, about July 15, 1928. The towers were completed, ready for cable operations, during June, 1929. At that time the contractor for the cable construction had made a beginning on the installation of his equipment. Grillages under the saddles on top of the towers were erected on the towers early in July, and the erection of the spinning equipment, including the steel superstructure on the towers, and the footbridges, proceeded during the months of July, August, and September. Cable spinning was begun October 21, 1929, and was completed August 8, 1930, after which the suspender ropes were placed in time to permit the erection of the first suspended steelwork on October 28, 1930. Nine weeks later. December 29, 1930, the last panel of main-span steelwork was in place. The side-span floor steel erection, which proceeded more slowly, was completed on January 21, 1931.

Contractors and Personnel

The towers and floor steel were supplied and erected by the McClintic-Marshall Company at a cost of $10 823 000. For the contractor, David S. Gendell, Jr., M. Am. Soc. C. E., Manager of Erection, was in charge, assisted by Mr. A. S. Halteman, as Resident Engineer. Mr. H. G. Reynolds was Foreman for the work on the New York side and Mr. W. B. Fortune on the New Jersey side.

The cables, suspenders, and anchorage steelwork were fabricated and erected by the John A. Roebling's Sons Company at a cost of $12 193 000. Mr. C. C. Sunderland was Chief Engineer for the Contractor, and Mr. C. M. Jones, Assistant Chief Engineer. W. E. Joyce, M. Am. Soc. C. E., served as Resident Engineer and Mr. W. R. Coppage as Superintendent.

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GEORGE WASHINGTON BRIDGE: MATERIALS AND FABRICATION OF STEEL STRUCTURE

BY HERBERT J. BAKER,[1] ESQ.

SYNOPSIS

The main structure of the George Washington Bridge, because of its type and magnitude, required for its construction large quantities of metallic materials of various kinds.

To describe the methods of manufacture and fabrication followed in the mills and shops on the various parts of the metallic structure, to record the properties of the materials incorporated in the structure, and to outline the methods of testing that were followed in order to insure conformity with the specifications, are the objects of this paper.

No material in this structure is new and untried. Except for modifications in the requirements for yield point of silicon steel in the towers and for tensile strength and yield point for heat-treated anchorage eye-bars and cable wire, like materials have been manufactured in the past to similar specifications.

The most modern methods of manufacture were followed in the various operations. These operations are of special interest because of the large quantities of materials involved and because of the unusual size of members.

INTRODUCTION

The four principal divisions of the main steel superstructure, listed in the order of their construction, are the anchorages, towers, cables, and suspended structure. Each of these divisions contains a number of different kinds of material.

In the anchorages are structural steel girders to which the cables are connected by chains of heat-treated eye-bars. The lower links of eye-bars are connected to the girders by heat-treated carbon-steel pins. The remaining pins of the eye-bar chains are made of annealed carbon steel. The cables are connected to the eye-bars by means of cast-steel strand shoes.

[1] Engr. of Inspection, The Port of New York Authority, New York, N. Y.

The towers are made of rolled structural steel. The cables are constructed of cold-drawn wire. They are supported at the towers and at the turning points on the anchorages by cast-steel saddles. The cables are wrapped with soft galvanized wire at points not otherwise covered. The tower and anchorage saddles rest on annealed carbon-steel rollers and rockers, respectively. The cable bands, which are made of cast steel, are clamped to the cables by heat-treated carbon-steel bolts.

Wire ropes, to which are attached cast-steel sockets, are the means by which the floor system is suspended from the cables. The floor system is made of rolled structural steel. In this part of the structure are miscellaneous pins of annealed carbon steel; miscellaneous cast-steel members; cast phosphor-bronze bushings; and rolled manganese-bronze wearing plates.

The materials are treated under the following headings: (1) Heat-Treated Eye-Bars; (2) Structural Steel; (3) Cast Steel; (4) Cable Wire; (5) Suspender Rope; (6) Other Materials; and (7) Distribution of Materials in Main Divisions of Structure.

Materials were inspected in various stages of manufacture. For this work, a force of men, skilled in the inspection of the various kinds of materials, was organized. In addition to the inspection at the place of manufacture, samples were analyzed by the Port Authority in its laboratory. For the analytical work a Chief Chemist, experienced in analyzing metallic materials, and the necessary assistants were employed. The personnel (a maximum of forty men) engaged in inspection and analytical work on this and other Port Authority projects was under the direction of the writer.

All physical tests upon which the acceptance of the materials was based were made by the manufacturers at their plants. These tests were witnessed by the Port Authority inspectors. In addition to the acceptance tests, a number of "check" tests were made by the Port Authority in its laboratory as special studies. The manufacturers were required by the specifications to furnish the check test specimens, machined and ready for testing.

(1) Heat-Treated Eye-Bars

Eye-bars were used only in the anchorages of the structure. They are 10 in. wide, but variable in the other two dimensions, being $1\frac{5}{16}$ in., $1\frac{3}{4}$ in., or $1\frac{7}{8}$ in. thick and from 35 ft $3\frac{3}{32}$ in. to 38 ft $7\frac{7}{32}$ in., between centers of pin-holes. For the two anchorages 3 456 bars were used.

The function of the anchorage eye-bars and the considerations governing the choice of material are treated in the paper on the design of the superstructure.[2] A heat-treated material with minimum values for yield point and tensile strength of 50 000 and 80 000 lb per sq in., respectively, and a total elongation of at least 8% in 18 ft, was specified. It was further stipulated that any twelve consecutive tests should show a minimum average yield point of 53 300 lb per sq in., and a minimum average tensile strength of 85 000 lb per sq in., respectively.

[2] "George Washington Bridge: Design of Superstructure," by Allston Dana and Aksel Andersen, Members, Am. Soc. C. E., and George M. Rapp, Assoc. M. Am. Soc. C. E., see p. 119.

Making and Rolling the Steel.—The steel was made and rolled into flats of the width and thickness desired in the eye-bars, at the Homestead, Pa., plant of the Carnegie Steel Company. The only values specified for the untreated material were for the chemical properties of phosphorus and sulfur, the content of which was not to exceed 0.04% and 0.05%, respectively. The material furnished was a carbon steel of the following average analysis: Carbon, 0.35%; manganese, 0.60%; phosphorus, 0.022%; and sulfur, 0.035 per cent. It was made by the basic open-hearth process.

All ingots were made by pouring the metal in the top of the moulds. In some cases hot top moulds were used. The ingots, in general, had a cross-section of 27 by 32 in. and were bloomed to one of 16 by 16 in. In making the blooming-mill discard care was exercised to eliminate piping and harmful segregation. Slabs of two sizes were cut from the bloom from which was rolled either a single-length or a double-length eye-bar flat.

The slabs were rolled to eye-bar flats on a 42-in. Universal plate mill. This mill is equipped with a manipulator for turning the slab on edge as all rolling is done with the horizontal rolls until the slab has been reduced to a square section, each side of which is slightly greater than the width of the required eye-bar flat. The side or vertical rolls are then brought into play to obtain and maintain the desired width, and rolling is continued until the desired thickness is obtained. It should be noted that the 27 by 32-in. ingot was reduced, by rolling, about 98% in producing an eye-bar flat.

Physical properties of the material in the "as-rolled" condition were not specified. However, tensile tests on material from each melt met certain minimum standards set by the eye-bar manufacturer, namely, yield point and tensile strength of 35 000 and 70 000 lb per sq in., respectively, and an elongation of 20% in 8 in.

Manufacture of Eye-Bars.—The rolled eye-bar flats were shipped from the mill to the Ambridge, Pa., plant of the American Bridge Company for manufacture into eye-bars.

In the order of their occurrence, the mechanical operations conducted in manufacturing these bars consisted of forming the heads before heat treatment and of straightening the bars and boring the pin-holes after heat treatment.

The heads were formed by repeating the operations of upsetting and rolling until the correct shape and thickness had been attained. After completing this operation, a pin-hole about $\frac{3}{4}$ in. smaller than the bored diameter was punched in the head.

The heat-treating furnaces for hardening and drawing the bars were gas-fired and equipped with pyrometers. Live rolls were used to carry the bars into and through the furnaces. The bars were hardened one at a time, the quenching medium being water, in which they were immersed edgewise. The bars were drawn two at a time, and when removed from the furnace were allowed to cool, under natural conditions, in shop air.

The straightening operation was a major one as the bars were kinked and cambered edgewise and flatwise and warped by the heat-treating opera-

tions to such an extent that they all required straightening. Edgewise camber of 4 to 6 in. in 24 ft occurred frequently. The straightening operation was performed on cold bars by means of gag-press and rolls.

Scope of Tests.—Following the usual practice in regard to proving the quality of eye-bars, tests to destruction of full-sized heat-treated members were specified. The number of full-sized tests had to be approximately 5% of the number of bars made in the early stages of manufacture and had to average about 3% of the number required for the work. In addition to the full-sized test of an occasional bar, the Brinell hardness test was required to be made on every bar. Specimen tests were also made on heat-treated material, the specimens for which were machined from eye-bar flats.

In addition to the data secured from the aforementioned tests, it was desired to determine the stress-strain characteristics of full-sized eye-bars. Therefore, tests to destruction were performed on two full-sized members in such manner as to obtain this information.

Brinell Hardness Test.—The need for a test to indicate the strength of each bar may be appreciated when the method of heat treatment is considered. The bars were heated and quenched one at a time; they were drawn two at a time. Obviously, a full-sized test does not truly represent the bars in any lot, nor does it indicate particularly a possible inclusion of other grades of material. The hardness test was used to overcome this deficiency.

The test conformed to Specifications of the American Society for Testing Materials entitled "Tentative Methods of Brinell Hardness Testing of Metallic Materials (Serial Designation E 10-27)." Accordingly, after the bar had cooled to shop temperature, a load of 3 000 kg was applied for 20 sec through a hardened steel ball 10 mm in diameter. As a rule, the impression was made at the approximate center line in three places—at the middle and near each head—on one side of those bars not used for full-sized tests, but at intervals of about 2 ft on both sides of the test bars.

The results of the Brinell hardness test were to be considered merely as a rough measure of the strength of the individual bars. However, the full-sized tests indicated that a range of average hardness numbers between 183 and 212, inclusive, could be considered as a reliable indication of the physical properties desired in the eye-bars. The Brinell hardness testing machine was not calibrated for this work because it was used only as a means of comparing the hardness of similar members. This fact should be borne in mind when considering the hardness numbers referred to herein.

Full-Sized Acceptance Tests.—To gauge the quality of the eye-bars not tested to destruction is, of course, the purpose of the full-sized test. Consequently, the selection of the bar to serve as a test specimen becomes a matter of prime importance. The choice should necessarily be governed by the history of the bar in order to acquire results indicative of the quality of other members showing approximately the same record as to chemical properties and details of manufacture. When, by some means, the indicated quality of the bar not tested to destruction is confirmed, then its quality can be readily accepted.

In this work, the bar's identity, a detailed record of its manufacture, and its Brinell hardness were available. To make the hardness test a dependable means for measuring the strength of an eye-bar or, in other words, to ascertain the relation between the hardness and other physical properties in order to estimate the strength of other bars, specimens differing from one another in average hardness, and others showing wide range in hardness within themselves, were tested to destruction.

Instead, therefore, of selecting bars for use merely as samples of lots containing a certain number of bars (in other words, at random, as was specified), they were chosen generally with the view of investigating the effects, on the physical properties, of a number of variable factors that were noted in the manufacture from steel mill to finished product. A list of these factors would include the chemical properties; position that the bar occupied in the ingot (as top, center, or bottom); surface imperfections (as scores or guide marks); quenching or drawing temperatures, or a combination of both; re-quenching; re-drawing; re-forging followed by re-treating; hardness numbers and uniformity of hardness throughout length; and excessive straightening, particularly removal of short local kinks or twists. The effects of one of these factors (chemical properties), are predictable and could be confirmed by the Brinell hardness test supported by data gathered from test bars as the manufacturing progressed. The extent to which other factors affected the desired physical properties of the bars could be ascertained only by a full-sized test.

A total of 111 full-sized tests were made. Two of these tests gave results outside the specified requirements, the elongation in 18 ft being about 50% of the minimum specified. The records of the two test bars disclosed nothing that would indicate to what these failures might be attributed. The average results of the remaining 109 tests are given in Table 1. All specimens broke in the body of the bar.

TABLE 1.—RESULTS OF 109 FULL-SIZED EYE-BAR TESTS

	Minimum	Average	Maximum
Yield point, in pounds per square inch	50 900	57 700	63 300
Tensile strength, in pounds per square inch	80 200	89 300	97 700
Percentage elongation, in 18 ft	8.5	10.8	13.9
Percentage reduction of area	25.6	45.4	54.5

The full-sized members were tested in an hydraulic type of testing machine at the Ambridge plant of the American Bridge Company. This apparatus was calibrated against the Emery testing machine at the National Bureau of Standards, U. S. Department of Commerce, by the use of two 10 by $1\frac{3}{4}$ in. by 16-ft eye-bars. The results of the tests indicated that for the same elongation the eye-bar testing machine loads were slightly higher than those of the Emery testing machine at the Bureau of Standards, but the error has been given no consideration in the recorded results of full-sized eye-bar tests.

The two bars used for calibration purposes were tested later to destruction in the eye-bar testing machine. The results and description of this test are given subsequently under the heading, "Stress-Strain Tests of Full-Sized Bars."

Specimen Tests.—Studies were made, by means of specimen tests, to determine the uniformity in physical properties of the material throughout the cross-section of heat-treated eye-bar flats.

The specimen material, two 12-ft flats, one from each of two open-hearth melts, was heat-treated in the same manner and with the same equipment as the eye-bars. The axes of the specimens were parallel with the longitudinal axes of the flats, and from the 10-in. width of each flat, nine specimens, equally spaced, were obtained. The specimens were machined to a diameter of 0.505 in. with a gauge length of 2 in.

The average tensile strength and yield point of the material (as revealed by the tests of the two specimens machined from the edges of one flat) were 103 000 and 67 500 lb per sq in., respectively. The corresponding average for the remaining seven specimens from this flat were 85 400 and 50 800, ranging between 83 700 and 88 000 for tensile strength and between 49 000 and 53 000 for yield point. The tests of the specimens from the other flat, while differing somewhat in absolute values, showed very similar variations in physical properties between the material at the edge of the flat and in the body of the cross-section. The average modulus of elasticity of the material, all tests considered, was found to be 29 900 000 lb per sq in., ranging between 28 600 000 and 30 300 000 lb per sq in., which corresponds with the value of 29 700 000 lb per sq in. for this property obtained on the tests of full-sized bars.

Stress-Strain Tests of Full-Sized Bars.—The results of special tests made for the purpose of ascertaining the characteristics of full-sized heat-treated eye-bars under tension are plotted in Fig. 1, and arranged for comparison in Table 2.

TABLE 2.—LOADS AND ELONGATIONS, FULL-SIZED, HEAT-TREATED EYE-BARS UNDER TENSION (SEE FIG. 1).

Load, in thousands of pounds	Total Inches of Elongation,* in a Gauge Length of 100 Inches		Load, in thousands of pounds	Total Inches of Elongation,* in a Gauge Length of 100 Inches		Load, in thousands of pounds	Total Inches of Elongation,* in a Gauge Length of 100 Inches	
	Test 1055*A*	Test 1055*B*		Test 1055*A*	Test 1055*B*		Test 1055*A*	Test 1055*B*
	Dial Readings			Dial Readings			Scale Readings	
100...	0.0000	0.0000	975....	0.1731	0.1770	1 225....	1.475	1.805
200...	0.0184	0.0186	1 000....	0.1823	0.1855	1 250....	1.660	1.990
300...	0.0386	0.0372	1 025....	0.1909	0.1965	1 275....	1.825	2.205
400...	0.0568	0.0563	1 050....	0.2052	0.2115	1 300....	2.025	2.405
500...	0.0754	0.0750	1 075....	0.2404	.....	1 325....	2.225	2.670
550...	0.0852	0.0853	1 100....	0.4879	.....	1 350....	2.415	2.915
600...	0.0942	0.0942	1 105....	0.7504	.....	1 375....	2.635	3.170
650...	0.1038	0.1039	1 105....	0.7694	.....	1 400....	2.900	3.445
700...	0.1134	0.1128				1 425....	3.150	3.760
750...	0.1220	0.1223		Scale Readings		1 450....	3.450	4.170
775...	.....	0.1262				1 475....	3.800	4.620
800...	0.1328	0.1313	1 075....	.....	0.895	1 500....	4.250	5.170
825...	.....	0.1364	1 100....	.....	0.980	1 525....	4.675	5.870
850...	0.1438	0.1411	1 105....	0.820	.....	1 550....	5.350	7.020
875...	.....	0.1467	1 125....	0.840	1.155	1 570....	.....	8.770
900...	0.1544	0.1530	1 150....	0.985	1.295	1 575....	6.150	.....
925...	0.1604	0.1578	1 175....	1.175	1.490	1 600....	7 350	.....
950...	0.1668	0.1650	1 200....	1.325	1.655	1 620....	10 000	.....

*Average readings top and bottom.

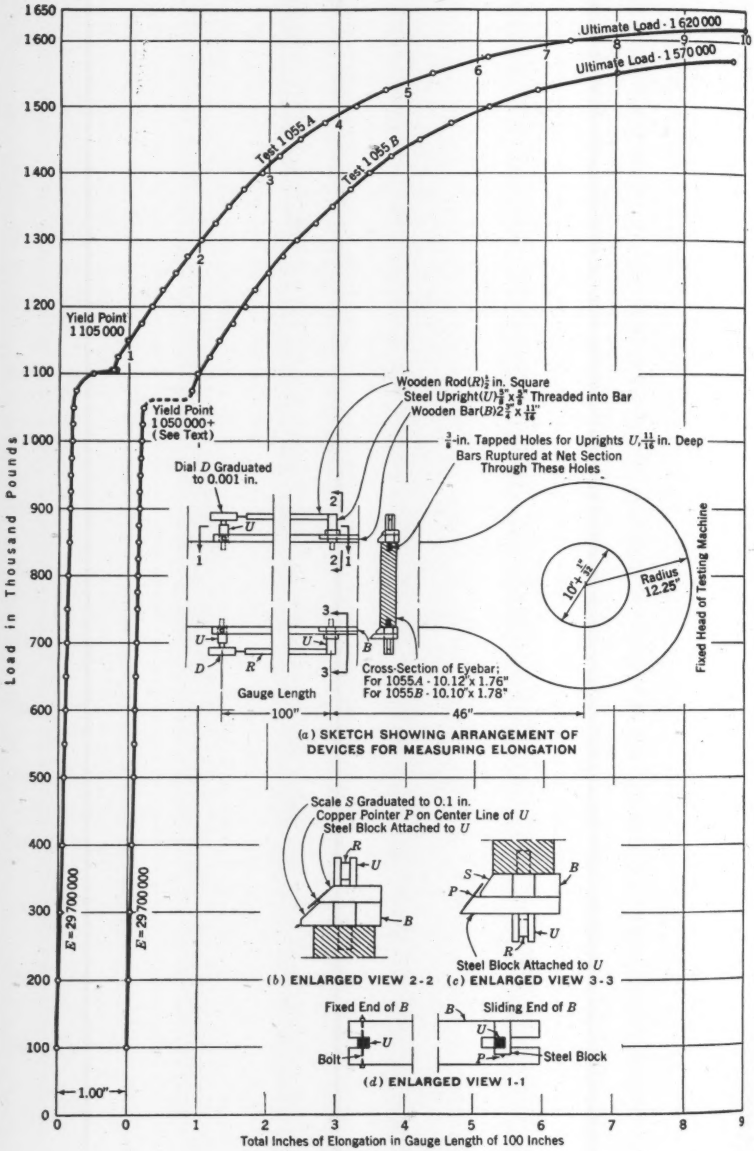

Load in Thousand Pounds
Total Inches of Elongation in Gauge Length of 100 Inches
Ultimate Load - 1 620 000
Ultimate Load - 1 570 000
Test 1055 A
Test 1055 B
Yield Point 1 105 000
Yield Point 1 050 000+ (See Text)
E = 29 700 000
E = 29 700 000
1.00"
Wooden Rod (R) ½ in. Square
Steel Upright (U) ⅝" x ⅝" Threaded into Bar
Wooden Bar (B) 2¼" x 11/16"
¾-in. Tapped Holes for Uprights U, 11/16 in. Deep
Bars Ruptured at Net Section Through These Holes
Dial D Graduated to 0.001 in.
Fixed Head of Testing Machine
Radius 12.25"
Gauge Length
100"
46"
Cross-Section of Eyebar;
For 1055A - 10.12" x 1.76"
For 1055B - 10.10" x 1.78"
(a) SKETCH SHOWING ARRANGEMENT OF DEVICES FOR MEASURING ELONGATION
Scale S Graduated to 0.1 in.
Copper Pointer P on Center Line of U
Steel Block Attached to U
(b) ENLARGED VIEW 2-2
(c) ENLARGED VIEW 3-3
Steel Block Attached to U
Fixed End of B
Sliding End of B
Bolt
Steel Block
(d) ENLARGED VIEW 1-1

FIG. 1.—LOAD ELONGATION CURVES OF FULL-SIZED EYE-BARS.

The apparatus used for measuring the elongation is also shown in a general manner in Fig. 1. The necessity for the two types of extensometers (*D* and *S*, in Figs. 1(*a*) and 1(*b*)) may be readily appreciated from consideration of the fact that the elongation beyond the yield point is rapid and extensive and that danger is present to observers and instruments. For these reasons, the scales were used to measure the large amounts of stretch after the dials had been removed. In performing the first test, which was made with Bar 1055*B*, the dials were removed quickly, after they had begun to spin in the yield-point region, because it was thought at the time that their capacity would be insufficient to indicate the stretch at the increment of load desired. Subsequent study of the data obtained from this test revealed the fact that the dials, which had a maximum movement of $1\frac{1}{2}$ in., could be used with safety in the yield-point region. The study also indicated that, with a change in the procedure of load application at the same point, more complete data might result in the second test. The effect of the different procedure is evident in the curve of Test 1055*A*, Fig. 1.

For taking the various readings to the yield point, an observer was stationed at each of the four gauges. For observing the stretch from the yield point and until rupture, the observers had stations on a crane moved to a point directly above the scales. The scales, made of ordinary cross-section paper and glued to the wooden bar, were read with the aid of binoculars which made the $\frac{1}{10}$-in. divisions readily visible.

The bars used for the tests were from the same open-hearth melt. Both showed average Brinell hardness numbers of 208, but the numbers for Bar 1055*A* ranged between 201 and 217 and those for Bar 1055*B*, between 179 and 217.

(2) Structural Steel

Materials.—Three grades of rolled steel—silicon, carbon, and rivet—were used in the construction of the main structure. The quantity of each kind (in tons), combining carbon with rivet steel, in the principal parts of the structure, is as follows:

	Silicon	Carbon
Towers	23 587	18 254
Anchorages and cables	.....	1 861
Suspended structure	8 132	11 210
Totals	31 719	31 325

The various grades had to meet the chemical and physical requirements given in Table 3. Referring to Column (5) in that table, the specifications for the towers provided that each group of ten melts of silicon steel (which, consecutively, must have met the individual minimum yield-point requirement), must have a minimum average yield point of 47 000 lb per sq in.

The elongation requirements in Item 8, Table 3, were modified as follows: For structural steel (Column (3)), deduct 1 from the percentage of elonga-

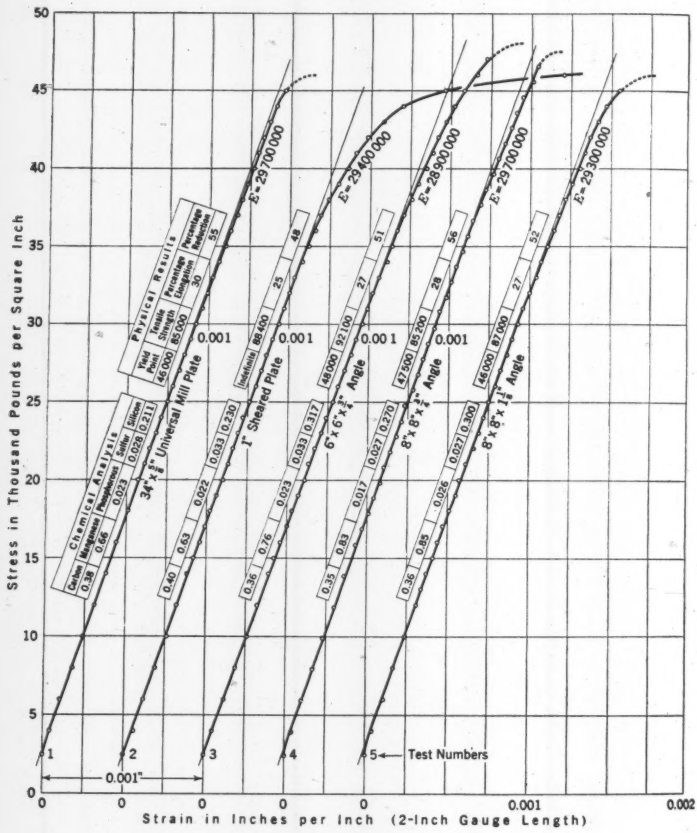


FIG. 2.—STRESS-STRAIN CHARACTERISTICS OF STRUCTURAL SILICON STEEL.

tion for every $\frac{1}{8}$ in. of thickness greater than $\frac{3}{4}$ in. (minimum elongation, 18%); for silicon steel (Column (5)), deduct 1 from the percentage of elongation for every $\frac{1}{4}$ in. of thickness greater than 1 in. (minimum elongation, 14%).

The area reduction requirements in Item 9, Table 3, were modified as follows: For structural steel deduct 1 from the value in Column (3) for every $\frac{1}{8}$ in. of thickness greater than $\frac{3}{4}$ in. (minimum reduction of area, 35%); for silicon steel, deduct 1 from the value in Column (5) for every $\frac{1}{8}$ in. of thickness greater than $\frac{3}{4}$ in. (minimum reduction of area, 24%).

The average properties of the materials incorporated in the structure, as determined by the standard mill tests, or acceptance tests and ladle analyses, are given in Table 4. In addition to the acceptance tests, check tests were

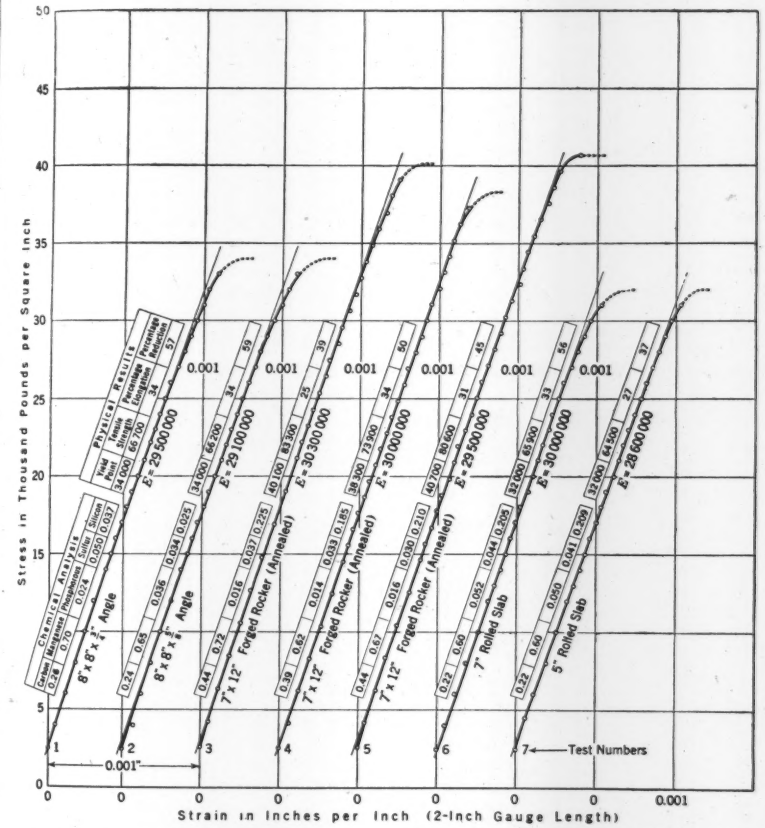

Stress in Thousand Pounds per Square Inch
Strain in Inches per Inch (2-Inch Gauge Length)
Test Numbers
0.001"
8" x 8" x 1" Angle
7" x 12" Forged Rocker (Annealed)
7" Rolled Slab
5" Rolled Slab
Chemical Analysis
Physical Results
E = 29 500 000
E = 29 100 000
E = 30 300 000
E = 30 000 000
E = 29 500 000
E = 30 000 000
E = 28 600 000

FIG. 3.—STRESS-STRAIN CHARACTERISTICS OF STRUCTURAL AND FORGED CARBON STEEL.

specified to be made with an extensometer on the usual specimen, $\frac{1}{2}$ in. in diameter. The average results of these tests, as well as the average results of the standard mill tests of the melts from which the check-test specimens were selected, are given in Table 5. This table shows clearly that the yield point of the material, as measured in the usual manner by the drop of the beam of the testing machine in commercial testing of standard mill test specimens, is appreciably higher than the value of this property as measured more accurately on machined specimens otherwise tested. It also shows that with respect to the tensile strength the two methods of testing give similar values. Typical stress-strain diagrams are shown in Figs. 2 and 3. The specimens were machined to a diameter of 0.505 in. Samples for the chemical analyses were milled from tested specimens. The broken-line extension of the curves

TABLE 3.—Chemical and Physical Requirements for Structural Steel

Item (1)	Description (2)	Carbon Steel		Silicon steel (5)
		Structural (3)	Rivet (4)	
	Chemical Properties:			
1	Carbon (maximum)	....	.....	0.40
	Phosphorus (maximum):			
2	Acid process	0.06	0.04	0.06
3	Basic process	0.04	0.04	0.04
4	Sulfur (maximum)	0.05	0.045	0.05
5	Silicon	....	.....	0.20 to 0.45
	Physical Properties:			
6	Tensile strength, in pounds per square inch	58 000 to 68 000	52 000 to 60 000	80 000 to 95 000
7	Yield point (minimum), in pounds per square inch	35 000	30 000	45 000
8	Elongation in 8 in. (minimum percentage)	1 500 000 / Tensile strength	1 500 000 / Tensile strength	1 500 000 / Tensile strength
9	Reduction of area (minimum percentage)	42	52	30
	Bend Test*:			
10	Material, $\frac{3}{4}$ in., or less; bend, 180°	Around $D = T$	Flat	Around $D = T$
11	Material more than $\frac{3}{4}$ to $1\frac{1}{4}$ in.; bend, 180°	Around $D = 1.5\ T$	.....	Around $D = 1.5\ T$

* D = inside diameter of bend; T = thickness of material.

indicates excessive stretch (not less than 0.009 in. per in.) before the next load increment had been reached. The yield point (the drop of the beam) was considered when plotting the broken lines.

TABLE 4.—Average Chemical and Physical Properties of Structural Steel

	Silicon Steel		Carbon Steel	
	Tower	Floor	Tower	Floor
Carbon	0.35	0.35	0.21	0.19
Manganese	0.78	0.76	0.50	0.52
Phosphorus	0.022	0.023	0.018	0.018
Sulfur	0.037	0.034	0.037	0.039
Silicon	0.27	0.27	.....	.....
Tensile strength, in pounds per square inch	88 800	86 400	63 600	63 400
Yield point, in pounds per square inch	50 800	51 300	38 200	39 400
Percentage elongation, in 8 in	22	22	28	29
Percentage reduction of area	43	44	52	54
Number of melts	895	257	1 134	484
Number of tests	1 884	669	2 207	971

The steel was made by the basic open-hearth process in a number of the larger steel plants in the eastern part of the United States. In general,

TABLE 5.—Comparison Between Average Check Test and Mill Test Results of Structural Steel

	Silicon Steel		Carbon Steel	
	Check tests	Mill tests	Check tests	Mill tests
Tensile strength, in pounds per square inch	88 000	86 900	63 500	64 300
Yield point, in pounds per square inch	47 400	50 700	34 900	39 500
Percentage elongation in 2 in	28	.....	35	.....
Percentage elongation in 8 in	.....	22	.....	29
Percentage reduction of area	53	41	62	52
Number of melts	146	146	87	87
Number of tests	150	347	87	189

current mill practice prevailed. However, one operation—cutting the rolled sections to the desired lengths—was a departure from standard practice.

The change in practice adopted by the mill materially assisted in keeping to a minimum the unsound angles shipped from the mills to the fabricating shops. In the past, the fabricators have been delayed and put to extra work because of the rejection at the shop of silicon-steel angles having the interior defect called a "pipe." The same is true, but to a lesser extent, in the case of carbon-steel angles. The pipe, unless present to an extreme degree, cannot be detected at the mill if the material is cut to the ordered length by the usual method of hot-sawing. It is prominently exposed, however, by shearing or milling operations. In view of this, it has become the practice to specify that all main material be sheared or milled on both ends as the first step in the shop work and was so specified for this work, but the fabricator recognized the advantage of eliminating the piped angles at the mill and as a consequence more than 75% of the angles were sheared before shipment to the shop.

Workmanship and Fabrication.—Workmanship of the most modern character in bridge construction was required.

Holes in all materials $\frac{3}{4}$ in. thick or less were punched with a die of a diameter $\frac{1}{8}$ in. smaller than the nominal size of the rivet and reamed after assembling $\frac{1}{16}$ in. larger than the nominal size of the rivet. Holes in material more than $\frac{3}{4}$ in. thick were drilled from the solid to the same diameter as sub-punched holes.

Material was reamed or drilled to full size with the parts assembled and firmly bolted together to prevent the accumulation of shavings or burrs between the separate parts of the member. After reaming or drilling, every hole was gone over with a special tool, and the sharp edges of the holes and the burr were removed to a fillet of about $\frac{1}{16}$-in. radius under each rivet head.

Sheared edges or ends of all exposed main materials and material thicker than $\frac{3}{4}$ in. were planed, faced, or chipped, so as to remove at least $\frac{1}{8}$ in. of material. The edges or ends of all other material were neatly sheared, but such edges were ground or chipped wherever they were rough, ragged, or irregular.

Rivets were driven by approved pressure tools wherever practicable. The speed and pressure of such tools were regulated to secure the best results in the work. Rivets were driven with pneumatic percussion hammers only when unavoidable and in such cases a pneumatic "bucker-up" was also used wherever possible.

All bearing surfaces were faced to a smooth surface, cut accurately to the proper angle with the axis of the member, so that the abutting members would have full and even bearing when properly aligned. The ends of the sections of the chords and columns were faced after they had been riveted with the exception of the projecting splice-plates. Where necessary, the sections were provided with suitable blocks or plates, spaced near each end to hold the plates and angles firmly in their proper relative position while the ends were being faced.

All field splices in the wind chords of the floor system and, with a few exceptions, the field splices in the columns of the towers, were assembled at the shop, then reamed or drilled with the abutting members in close contact and correct alignment, after which the splice material was dis-assembled, the burrs were removed, surfaces cleaned and painted, and the material was re-assembled. All other field connections were reamed or drilled to metal templates with hardened steel bushings set into accurately drilled steel plates.

The structural steel parts of the anchorages and the suspended structure were fabricated by the procedure more or less common for the types of members involved. The tower pedestals and column sections, on the other hand, are unusual in size and shape and, therefore, required exceptional methods of fabrication which are worthy of special mention. Accordingly, an outline is presented of the shop procedure used in the manufacture of these members by the three companies that supplied the materials for the tower proper. The American Bridge Company fabricated the members of the New York tower from the pier to about mid-height; the Bethlehem Steel Company, a like part of the New Jersey tower; and the McClintic-Marshall Company, the remainder.

Pedestals.—The pedestals are made of rolled carbon-steel plates and angles. They were designed with a vertical field splice in order to facilitate shipment, but only those for the New York tower were so fabricated. These members for the New Jersey tower were riveted in the shop to form a unit, differing from the other pedestals in one other respect, namely, the cap-plate was made of one piece instead of two.

The pedestals are of cellular construction as shown in Figs. 4 and 5. Before fitting, the top and bottom edges of the web-plates were planed to facilitate the setting of the cap and base angles. In this edge-planing an allowance of about $\frac{5}{16}$ in. in depth of plate was made to permit further planing subsequent to the operation of assembling. The same allowance for planing was made in the sheared length of the stiffener angles.

As previously mentioned, the fabricator of the New Jersey tower pedestals elected to plane and ship each pedestal as a unit. Accordingly, the cellular part of a pedestal was fabricated in two sections, as designed, after which they were fitted together and riveted at the vertical splice, the resulting assembly being handled as a unit through the subsequent operations performed in the machine shop. The top of this assembly was finished first on a boring mill, after which the member was turned upside down and finished on the bottom. The two-piece base-plate, fine finished on the top side and rough finished on the bottom to about $\frac{1}{8}$ in. more than the detailed thickness, was fitted and riveted. The pedestal was then finished to the detailed over-all height by planing the bottom of the base-plate. To complete the member, the one-piece cap-plate, already planed to size and drilled to template, was clamped in place and used as a template for drilling the holes in the cap angles. By this method of fabrication, a pedestal of the specified height was built, in which all stiffeners and web-plates were in full bearing with the cap and base-plates. The slight loss sometimes produced in the height

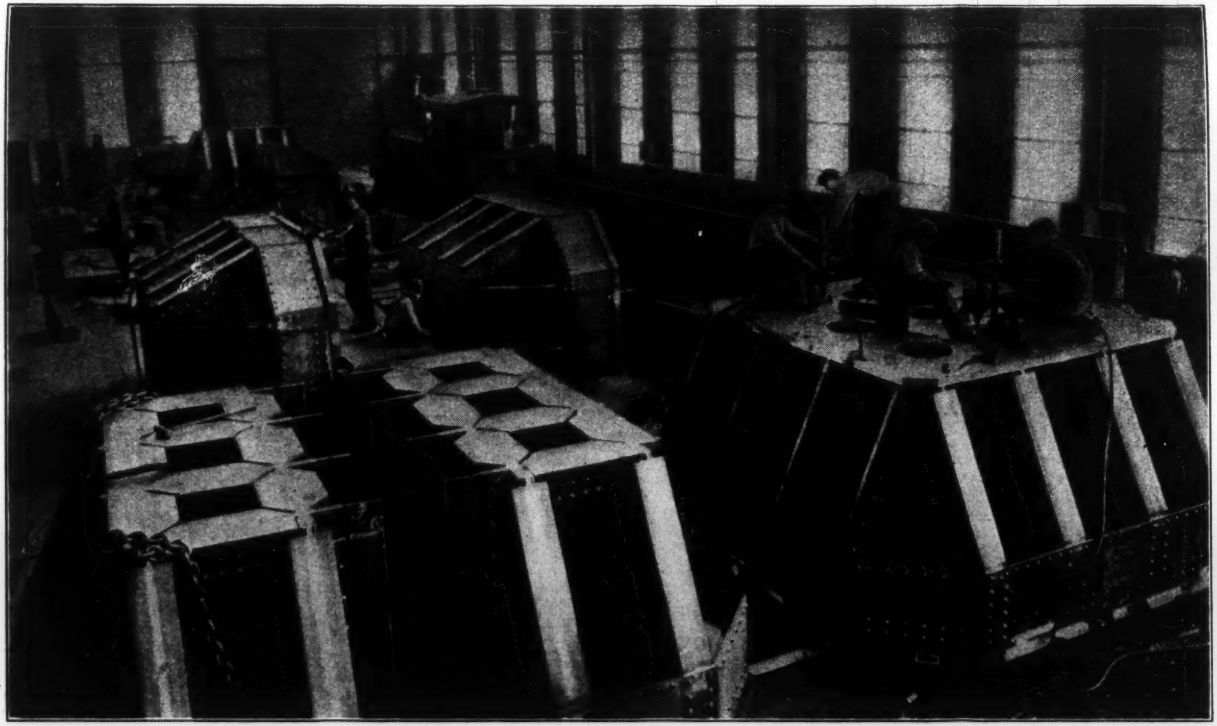


FIG. 4.—TOWER PEDESTALS IN PROCESS OF FABRICATION, GEORGE WASHINGTON BRIDGE.

of the cellular unit by excessive planing of any part to secure complete bearing surfaces was corrected merely by increasing the thickness of the base-plate.

The New York tower pedestals were fabricated and shipped as two-piece units to accommodate the shop equipment. The bed of the planer was not wide enough to support a complete pedestal, but it had sufficient length for two half-pedestals. In conducting the planing operations on these members the bottoms of the cellular units were first planed individually. In the next step of the procedure, two cellular units were placed in line on the planer bed and the tops planed at the same time, or as one piece. These two parts, of course, were of the same height and were treated as a unit in the succeeding operations which were somewhat similar to those followed on the New Jersey tower pedestals. In some cases, however, the finished height was made

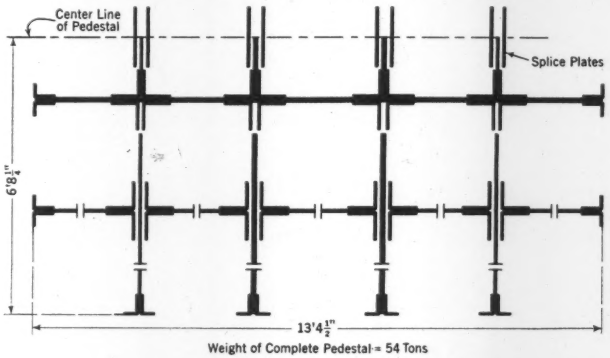


FIG. 5.—CROSS-SECTION OF HALF-PEDESTAL SHOWING FITTING OPERATIONS.

somewhat less than that specified in order to obtain complete bearing surfaces. This loss in height could not be corrected by the method followed on the New Jersey tower pedestals because the base-plates had been finished to size at the mill where adequate planing equipment was available. The field forces were notified of such error so that the members would be properly set.

Column Sections.—The column sections are made of rolled silicon steel plates and angles except those at the top of the outside columns, which are made of rolled carbon steel. The detail material is also of carbon steel except for the gusset-plates of the main bracing, which are of silicon steel. The column sections are one-piece members except some of the upper sections, which were designed with vertical field splices.

In general, the web-plates were sub-punched and the shaft angles were sub-drilled. Most of the material at the splices, both horizontal and vertical, was drilled from the solid while assembled and all other field connections were drilled full size through metal templates after the members had been milled.

Some difficulty was experienced in fitting the heavy gauge angles because the legs were not at a right angle to each other. The mill was not successful

in correcting this condition of the angles. In order to secure close contact of metal to metal—and therefore tight rivets—the parts had to be completely bolted. Illustrations of typical fitting operations are shown in Figs. 6 and 7. The sequence of the principal fitting operations as followed at one of the

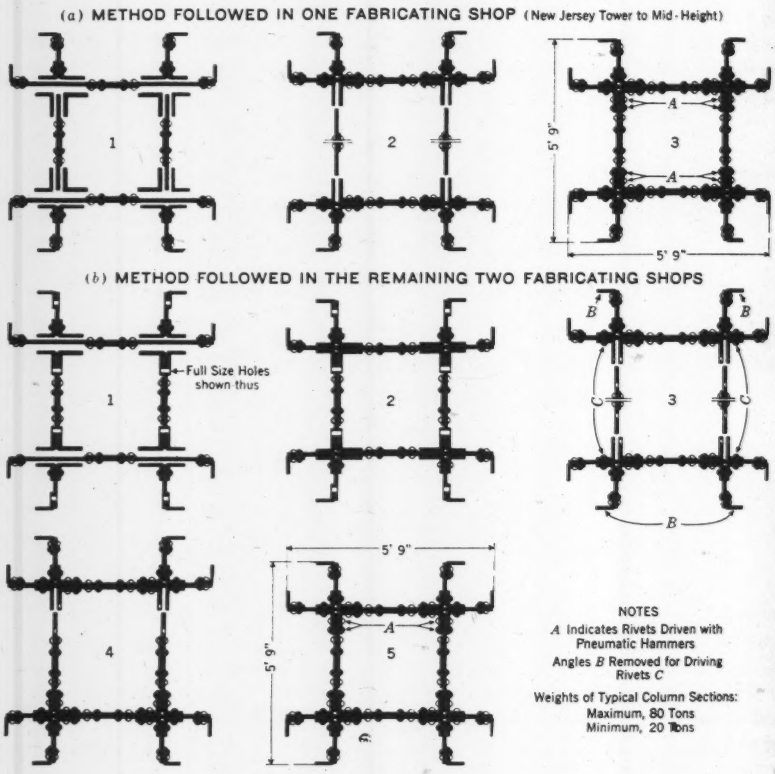


FIG. 6.—SEQUENCE OF PRINCIPAL FITTING OPERATIONS ON TYPICAL TOWER COLUMN SECTIONS.

fabricating shops is demonstrated by the three steps in Fig. 6(*a*). The remaining two fabricating shops followed the sequence illustrated by the five steps in Fig. 6(*b*).

In conducting milling operations on a column section careful consideration was given to the finished length checked by steel measuring bars. Any variation between the specified and measured lengths was included in the adjoining section. However, the need for correcting possible accumulation of errors from any cause to procure true and level seats for the grillages supporting the cable saddles was anticipated. Accordingly, the specifications

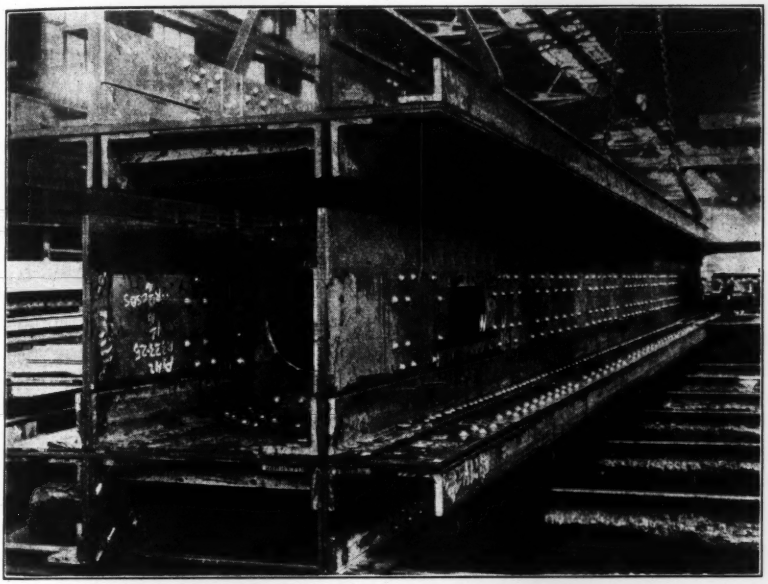


FIG. 7.—FINAL FITTING OPERATION ON TYPICAL TOWER COLUMN SECTION.

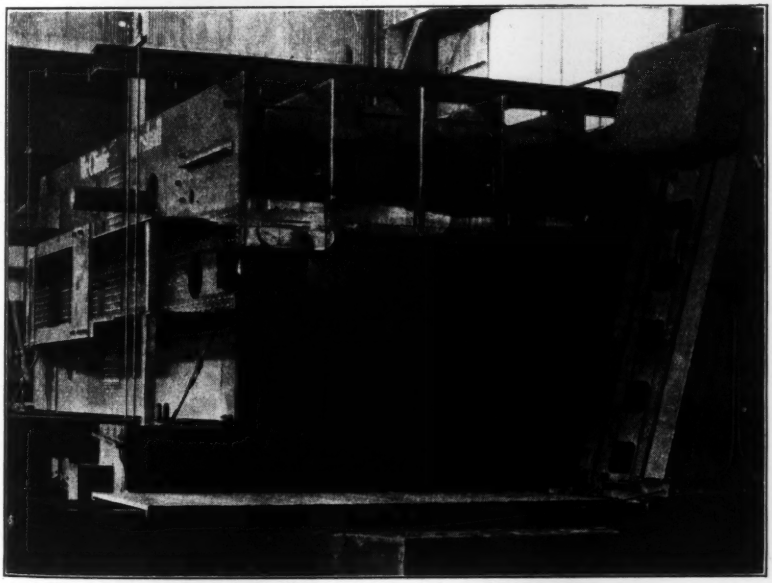


FIG. 8.—MILLING TOP SECTION OF TOWER COLUMN.

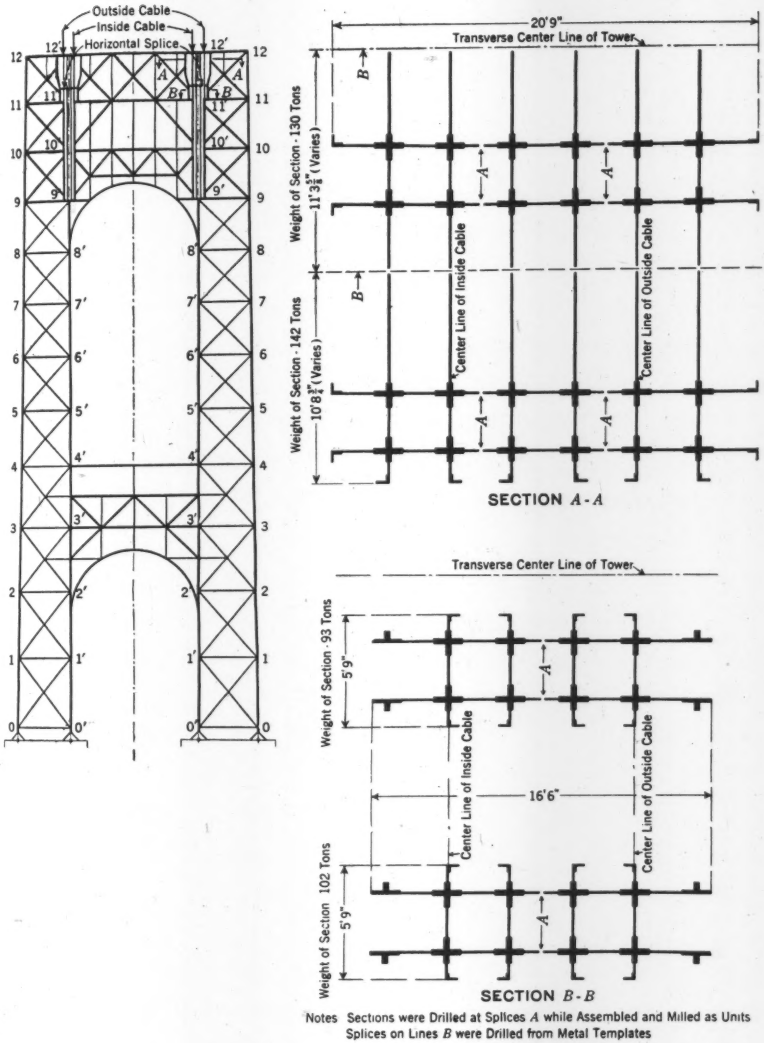

Outside Cable
Inside Cable
Horizontal Splice
20'9"
Transverse Center Line of Tower
Weight of Section - 130 Tons
11'3½" (Varies)
Weight of Section - 142 Tons
10'6½" (Varies)
Center Line of Inside Cable
Center Line of Outside Cable
SECTION A-A
Transverse Center Line of Tower
Weight of Section - 93 Tons
5'9"
16'6"
Weight of Section 102 Tons
5'9"
SECTION B-B
Notes Sections were Drilled at Splices A while Assembled and Milled as Units
Splices on Lines B were Drilled from Metal Templates

FIG. 9.—CROSS-SECTION OF UPPER COLUMN SECTIONS, SHOWING ASSEMBLIES FOR REAMING AND MILLING.

required that the top sections of the columns supporting the saddles directly, as well as the top sections of the remaining columns at the same level, be milled at the shop to lengths (determined by a field survey) that would bring the column tops of each tower to a plane. The field checks were made when the towers had been erected to the horizontal field splice, No. 10 (see Fig. 9), and from these data the lengths of adjoining sections were made to take up a maximum variation of $\frac{7}{32}$ in. between column heights in the New York tower and $\frac{3}{16}$ in. in the New Jersey tower. The method used in reaming the field rivet holes of the splices affected is described subsequently.

The work of milling the upper two sections of the eight inside columns of each tower was somewhat difficult inasmuch as they were large and heavy. Some sections were made of two pieces and some of three pieces, which were assembled as such and then reamed at the vertical field splices and, finally, milled as units (see Figs. 8 and 9). In view of the extreme dimensions of these upper sections, it was found necessary to install a special machine for the work. This machine was designed for milling both ends of a section at the same time. Fig. 10 shows the milling apparatus at one end of the machine.

The horizontal field splices (except No. 6 and No. 10, Fig. 9), were reamed or drilled while the sections were spliced together at the shop (see Fig. 11). The method that is usually followed in performing this operation is to sub-punch or sub-drill all splice-rivet holes in all main and splice material prior to fitting, and then to ream the holes after the splice has been fitted. This practice gives fair results when the extreme cross-sectional dimensions of the spliced members are small and the members are composed of only a limited number of pieces of main material. When these dimensions, as well as the number of pieces, are increased the results are too often unsatisfactory. This is because it is difficult, if not impossible, to assemble and rivet the various elements forming a member, in such a manner that when the member is milled the distance from the plane of the milled end to the groups of splice-rivet holes in the various webs is in agreement with the details. When two such members are spliced, the error in distance between the set of holes in the splice in one member and those in the other is quite often increased. Consequently, the splice material which previously had been sub-punched or sub-drilled according to detail would not fit these conditions, and imperfect holes would result when they were reamed to full size.

Two methods were used to obtain good holes in the horizontal splices of the columns. At two plants the following practice was adopted: A few holes in the main material and corresponding holes in the splice material on the inside of the member were sub-punched or sub-drilled to permit assembling them. All holes were sub-punched in that splice material which was on the outside of the member and which served for use as templates in drilling the full-sized holes. At the other plant all holes were sub-punched in the splice-plates and sub-drilled to metal templates in the main material and splice angles. However, the holes in the main material were not sub-drilled until after the members had been milled, so that advantage could

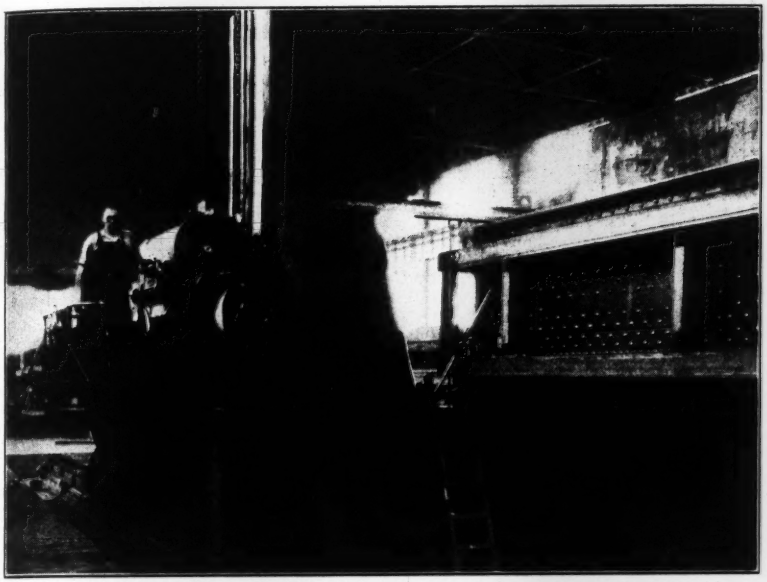


FIG. 10.—VIEW SHOWING ONE END OF LARGE MACHINE FOR MILLING TOP SECTIONS OF TOWER COLUMNS.

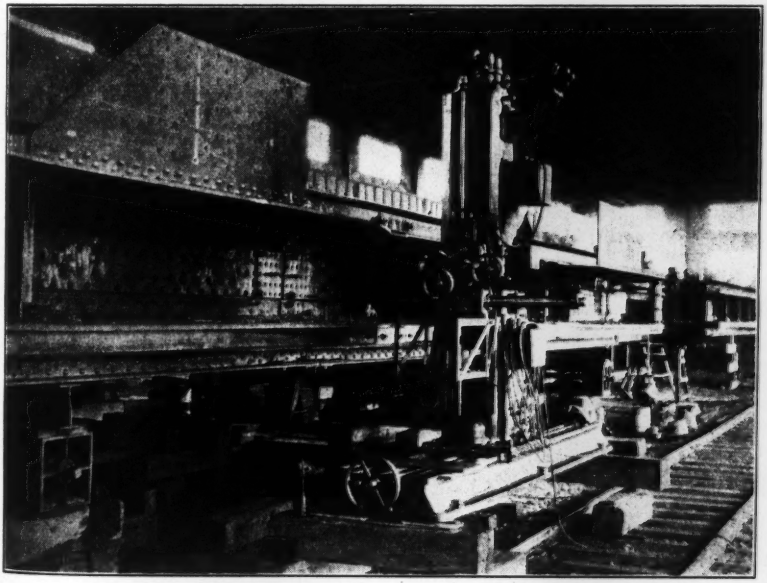


FIG. 11.—REAMING HORIZONTAL FIELD SPLICE OF COLUMN.

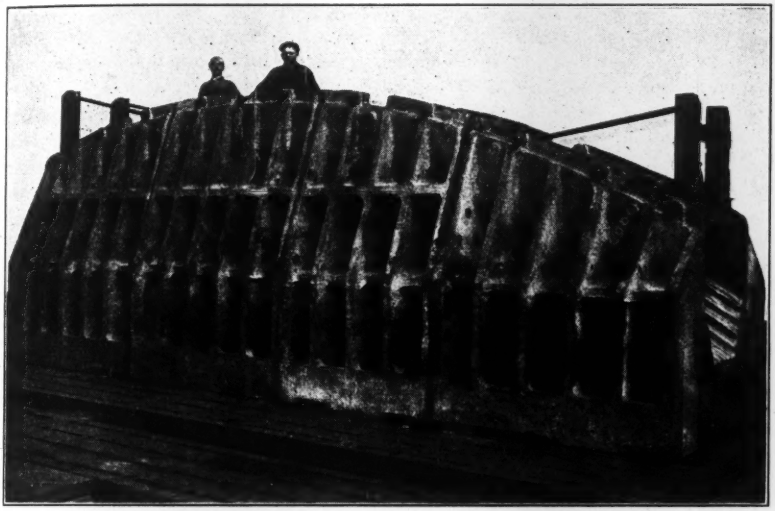


FIG. 12.—VIEW OF MAIN TOWER SADDLE.

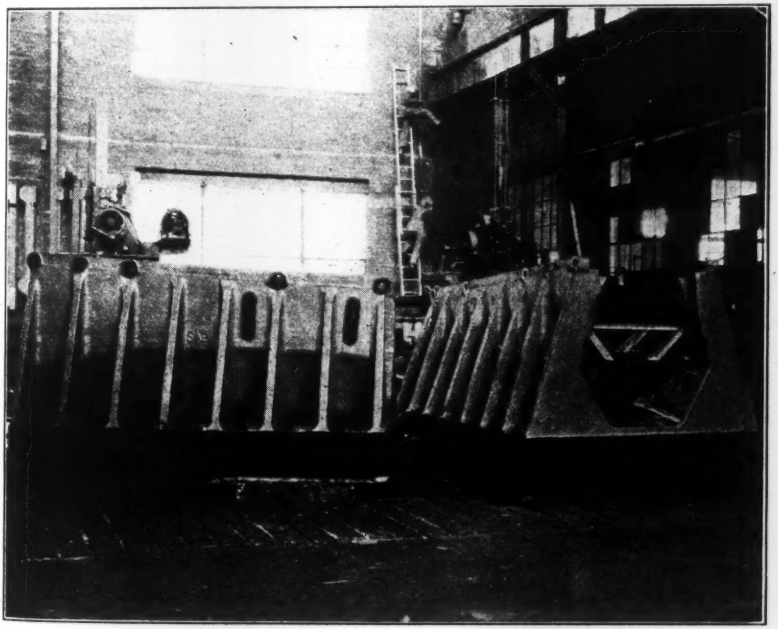


FIG. 13.—VIEW OF ANCHORAGE SADDLES.

be taken of milled ends for setting templates. After the splice was properly fitted, the holes were reamed to full size. Very satisfactory results were obtained by these two methods of conducting the work.

In the case of the horizontal splice, No. 6, where the column sections fabricated at different plants are spliced, and No. 10, where corrections in column lengths were made, the field-rivet holes in the upper halves of the splices were reamed to metal templates. These holes of Splice No. 6 were reamed to a diameter of $\frac{1}{8}$ in. in the shop and to $1\frac{1}{8}$ in. in diameter, or full size, in the field after the connections had been pinned and bolted. In Splice No. 10 the holes were reamed full size at the shop. The holes in the lower halves of these two splices were reamed as described for the typical splices.

(3) Cast Steel

Castings of unusual size are in service in the main structure. Some idea of their magnitude may be drawn from the average weights of individual pieces. A tower saddle averages 359 400 lb in weight and is composed of four segments, as shown in Fig. 12, of which the heaviest is 110 240 lb and the lightest, 74 370 lb. An anchorage saddle, however, is one piece of 43 670 lb (see Fig. 13). A cable band consists of two pieces semi-circular in shape, the longest band weighing 6 500 lb and the shortest, 1 150 lb. The total cast steel in the main structure weighs 2 643 tons.

To secure the desired quality and workmanship in the cable bands, strand shoes, and other groups of smaller castings, the foundry followed methods of moulding and casting used in other products of similar shapes; and the machine shop particularly experienced the least difficulties inasmuch as the machining operations were comparatively simple. On the other hand, to secure sound and accurately machined castings for use as tower and anchorage saddles the foundry and machine shop were confronted with various problems because of the size and design of the members. Consequently, aside from the properties of the material, only an account of the foundry and machine work on the two kinds of saddles will be presented in this paper.

The main tower saddles were manufactured by the Midvale Company and the bulk of the remaining cast-steel members by the Wheeling Mold and Foundry Company.

Material.—The cast steel was made by the acid open-hearth process except for the suspender-rope sockets and some of the miscellaneous members for which electric-furnace steel was used. The requirements to which the material had to conform are given in Table 6. In this table are also shown the average properties of the cast steel as determined by the foundry or acceptance tests and the ladle analyses.

The number of test coupons (which were cast integrally with the members when possible) depended on the size of the member. Their location was controlled by the design of the piece and, on the saddles, particularly, by the position of the piece in the mould. The coupons generally were 1 in. by 5 in. in cross-section and 6 in. in length, but those on the saddle segments measured 2 by 6 by 8 in. Each of the segments—due to their size and

TABLE 6—Specified and Average Properties of Cast Steel

Part of structure	Number of tests	Physical				Chemical				
		Yield point, in pounds per square inch	Tensile strength, in pounds per square inch	Percentage elongation in 2 inches	Percentage reduction of area	Carbon	Manganese	Phosphorus	Sulfur	Silicon
Specified requirements		35 000 min.	65 000 min.	20 min.	30 min.	....	....	0.06 max.	0.05 max.	....
Main tower saddles	64	39 000	72 300	26	41	0.31	0.61	0.034	0.039	0.34
Anchorage saddles	16	39 500	75 200	27	41	0.34	0.65	0.044	0.032	0.34
Cable bands	659	37 800	71 000	30	49	0.29	0.69	0.040	0.029	0.33
Strand shoes	510	38 800	75 500	27	41	0.30	0.73	0.039	0.032	0.34
Suspender rope sockets *	85	41 600	69 700	28	43	0.25	0.63	0.036	0.040	0.29
Miscellaneous †	64	43 200	76 300	26	39	0.25	0.69	0.034	0.044	0.40
Average	..	38 700	72 900	29	45	0.28	0.67	0.038	0.035	0.34

* Electric furnace steel. † Includes both acid open-hearth and electric-furnace steel.

the fact that they were cast with the base horizontal and in the cope, or the top of the mould — had eight coupons spaced at intervals on both ribbed sides, from which two were selected to provide two tension and two bend test specimens. Affording a similar choice, each anchorage saddle had one coupon in the vicinity of each corner of the base, these members being cast with the base vertical. A half-cable band carried two coupons attached to the bore side. Weight governed the number of coupons on bands as well as on other castings, thus providing one tension and one bend test for each casting of 500 lb, or more, and sufficient specimen material for two tests of each kind per melt and annealing charge for lighter castings.

Foundry Work.—The two foundries in which the different saddles were made followed practically the same pouring, cleaning, and annealing procedure. The molten metal was introduced at the lowest point of the mould; the castings were cleaned by means of pneumatic chipping hammers and sand-blasting; and the annealing operation was conducted, of course, with the view of obtaining uniform structure throughout the castings in the charge.

The methods of moulding the two kinds of saddles, however, differed widely. The tower saddle segments were moulded in sectional flasks with the aid of patterns and dry sand cores. (See Fig. 14.) Previous to the operation of setting the cores the mould was dried by coke and coal fires lowered into place. The segments were cast upside down in respect to their position in the structure so that, with the introduction of the molten metal at the lowest point of the mould, the dirt and slag tended to float to the base of the segment where removal of contaminated metal was most practical. Therefore, an allowance was made in moulding to provide 1 in. of excess metal on the base, but this was increased subsequently to 3 in. to insure elimination of all unsound metal by machining.

Fig. 15 shows a segment of a tower saddle in process of cleaning. The segment is resting on one end with the cable groove to the right and the

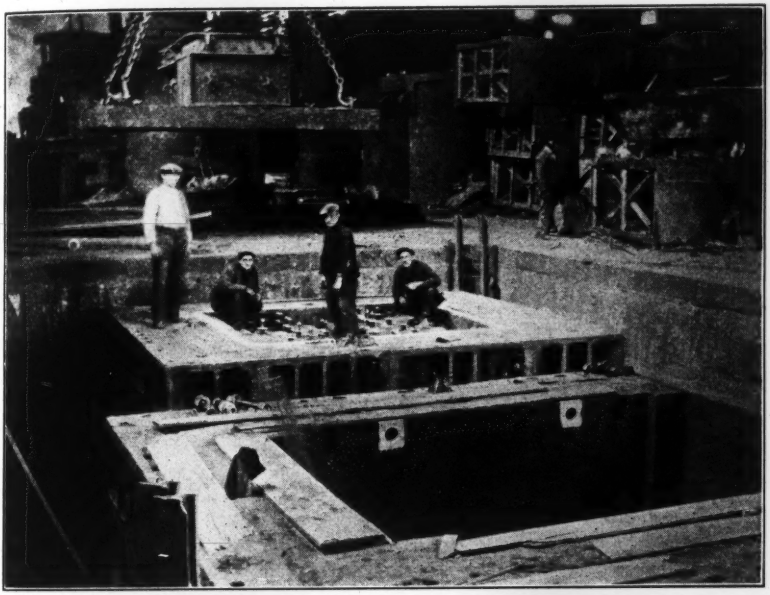


FIG. 14.—MOULDS FOR TOWER SADDLE SEGMENTS. IN FOREGROUND, MOULD READY TO RECEIVE CORES; IN BACKGROUND, CLOSING MOULD; "COPE COVER" IN CRANE HOOKS.

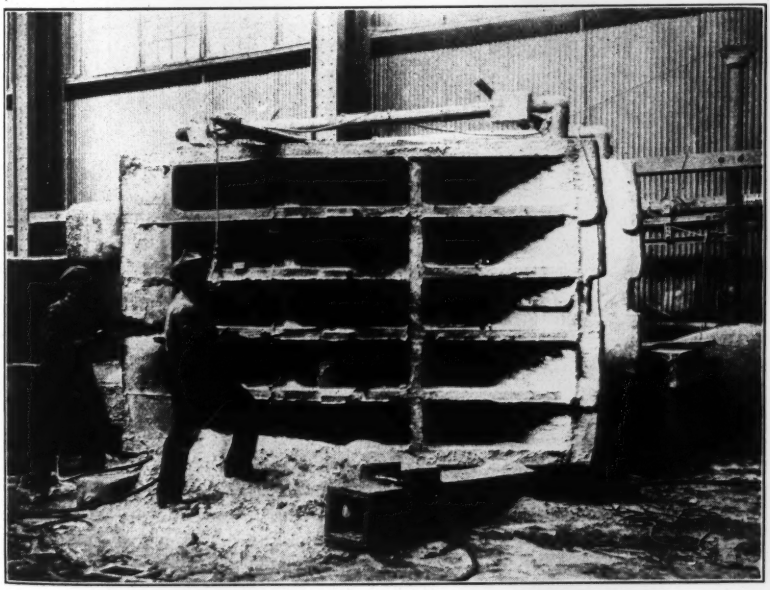


FIG. 15.—SEGMENT OF TOWER SADDLE IN PROCESS OF CLEANING.

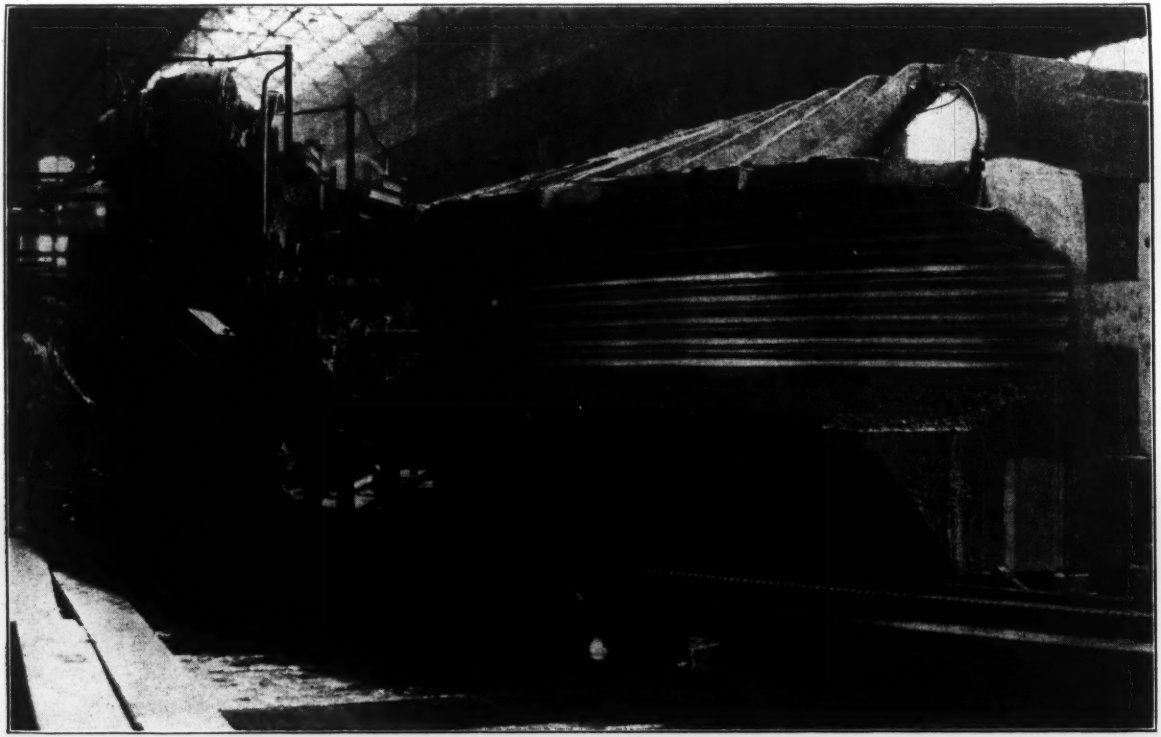


FIG. 16.—MACHINING CABLE GROOVE OF MAIN TOWER SADDLE.

base to the left. The gates, or the points where the metal entered the mould, are clearly indicated, being the points on either side of the cable groove where the runners are attached to the casting. The two risers, provided to "feed" the casting during solidification, are shown attached to the base; the test coupons, attached to the ribs, are clearly in evidence.

In the case of the anchorage saddles, the mould was made in a pit by a method known as the "core assembly". The procedure consisted of forming a mould of dry sand cores and of ramming sand between the cores and the sides of the pit. The members were cast on end; in other words, with the base in a vertical position.

Machine Work.—The machine work on an anchorage saddle was confined chiefly to simple planing operations on the base surface and flanges, and to a dressing process on the cable groove. The latter, a grinding operation, removed the high spots the locations of which were determined by a metal template.

The work of machining a tower saddle was a more involved operation because of the intricate cable groove and the number of segments After milling the base of each segment with an allowance in thickness to permit further planing, the ends were planed to secure tight joints in the completed saddle. Because of planing-machine limits, only three segments of the four were bolted together and finished on the base at one time. To complete the planing work on the base, one section was removed and the fourth one added to the other end, the base of the latter being finished to the plane of the other two. Finally, after the four segments had been assembled into a saddle supported on its side, the cable groove was machined. (See Fig. 16.) During this operation the member remained stationary while two milling machines traveled on the same curved track to shape the cable groove. The finishing cuts were made by one machine only in order to obviate the ridge that might result from the slight variation in the cutting action of the two machines.

(4) Cable Wire

The four cables are composed of cold-drawn galvanized steel wire about 0.196 in. in diameter. They were constructed from long lengths of wire made continuous by means of splices. One such length as shipped from the factory to the bridge site would measure about 28 miles and would weigh approximately 7½ tons. The total quantity of wire used is 28 308 tons.

In view of the large tonnage involved (which is about four times that of any previous single order for this kind of material) and also the uniformity of the product, an account of the manufacture in some detail is warranted. All manufacturing operations from the melting of the steel to, and including, the reeling of the finished product were carried out at the same plant. In the principal operations the methods were similar, in general, to the practice found in the cable-wire industry.

This material was manufactured by the John A. Roebling's Sons Company in its plant at Roebling, N. J.

Chemical Properties.—The specifications permitted a maximum carbon content of 0.85% and a phosphorus and a sulfur content of 0.04% each, on ladle analysis. On check analysis of the finished or semi-finished product, an excess beyond these limits was permitted, as follows: 10% in the case of carbon; 25% in the case of phosphorus; and 25% in the case of sulfur. The amounts of the other elements were not indicated.

For determining the percentages of carbon, manganese, phosphorus, sulfur, and silicon in the steel, a ladle analysis and two check analyses were made on samples from each melt. The sample for the ladle analysis was drilled from a test ingot which was cast after about one-half the melt had been teemed or run into the ingot moulds. The two samples for the check analyses were drilled from different billets. One sample was always taken from the top billet, or the billet adjacent to the top discard of the ingot, where the greatest segregation would most likely be found, and the other usually from the bottom billet of another ingot.

The average, maximum, and minimum results of the ladle and check analyses of the 958 melts of steel from which the wire in the cables was manufactured, are given in Table 7.

The degree of uniformity of the steel throughout the melt may be deduced from the amount of each element present as disclosed by the two check

TABLE 7.—CHEMICAL ANALYSES OF CABLE WIRE

	LADLE					CHECK				
	Carbon	Manganese	Phosphorus	Sulfur	Silicon	Carbon	Manganese	Phosphorus	Sulfur	Silicon
Average.....	0.80	0.63	0.029	0.036	0.19	0.81	0.63	0.029	0.034	0.19
Maximum....	0.85	0.83	0.040	0.045	0.32	0.93	0.77	0.042	0.046	0.34
Minimum.....	0.76	0.50	0.020	0.025	0.11	0.72	0.44	0.021	0.022	0.07

analyses, the samples for which, as previously indicated, were selected with the view of proving the material in this respect. Of the melts showing the greatest variations in amount of each of the three elements considered by the specifications, fifteen showed a difference in carbon content ranging between 0.07 and 0.14%; fifteen, in the case of phosphorus content, between 0.003 and 0.015%; and a like number, in the case of sulfur content, between 0.006 and 0.019 per cent.

Manufacture.—The manufacturing process is a standard one for producing this kind of wire. The materials are combined and melted in open-hearth furnaces. The ingots are rolled into billets which, in turn, undergo further rolling into rods. The rods are patented, pickled and cleaned, coated and baked with drawing vehicle, and, finally, cold-drawn into wire. The wire then passes into the galvanizing unit where it is annealed, cleaned, galvanized, and wound on drums from which it is taken to be reeled for shipment.

The initial operations vary with the manufacturer in some of the details. For this wire, the steel was made in acid open-hearth furnaces in small quantities of about 35 tons, or about 40% less in amount than is the

practice in the ordinary steel plant where the quantity per melt is approximately 50 tons and more. The ingots were cast in hot top moulds to lessen and confine the pipe; they were 14 by 14 in. in cross-section and weighed about a ton. The billets were 4 by 4 in. in section, but, later, were rolled to 2 by 2 in. when additional equipment was placed in operation; their weight in both sizes averaged about 380 lb. The rods rolled from these billets were reduced from a nominal diameter of 0.360 in. to one of 0.192 in. by cold-drawing them through four dies progressively smaller in diameter.

Tests and Physical Properties.—The material was subjected to physical tests before it entered the galvanizing unit and after it had been galvanized. A bend test only was performed on the wire in the first-mentioned or "green wire" stage. The Preece or copper sulfate dip test for thickness of zinc coat, a bend test, and a tensile test determined the quality of the galvanized product. The Preece test was made on 5% of the coils, and all specimens passed it. In addition to these routine tests, investigations were conducted to determine the modulus of elasticity of the wire and to ascertain the degree of uniformity in strength throughout the coil.

Bend tests were made on specimens from one end of every coil or length of "green wire" and from both ends of 10% of the galvanized coils. The test consisted of coiling the specimen for one complete turn on a mandrel $1\frac{1}{2}$ times the nominal diameter of the wire. No failures occurred in the "green wire" coils, and the number of coils which failed after galvanizing was negligible.

The specifications for the tensile test of the galvanized material required determinations of the yield point, tensile strength, and total elongation on both ends of 10% of the coils, but only of the latter two characteristics on both ends of other coils. The minimum values, respectively, for these properties were set at 150 000 and 220 000 lb per sq in. and 4% in 10 in. Furthermore, twelve consecutive tests had to show a minimum average yield point of 153 300 and a minimum average tensile strength of 225 000 lb per sq in. The former was defined as the stress that produced a stretch of 0.7% in 10 in. The cross-sectional area upon which the unit stresses are based, is derived from the gross diameter which includes the galvanizing.

The tensile tests presented a minor problem in load application which had to be solved at the beginning, because of the curve in the specimen to which the extensometer had to be attached. The specimens were curved to a radius of about 3 or 4 ft, or to a radius somewhat greater than that of the drums on which the wire had been wound. Obviously, such a condition affects the degree of accuracy with which the yield point may be measured. Experiments disclosed the fact that under a load of about 650 lb, and with a distance of 24 in. between the jaws of the testing machine, the curve is practically eliminated; that is, the 12-in. central portion of the specimen appeared to be straight as observed by the aid of a straight-edge. Furthermore, other preliminary tests furnished a value for the modulus of elasticity of 28 000 000 lb per sq in. for the material. Based on these data, a procedure was established for the routine yield-point tests in which an initial load of 850 lb was applied to the specimen before the attachment of the two

extensometers. The initial load of 850 lb was calculated to produce, for all practical purposes, an elongation of 0.1% for gross diameters within the specified limits of 0.192 in. and 0.200 in. The calculated percentage of stretch in these tests is more nearly correct when the diameter of the specimen is 0.196 in. which happens to be the average diameter of the wire actually incorporated in the cables. In the other tests, where only the tensile strength and total elongation were desired, an initial load of about 2 000 lb was applied before attaching one extensometer, but no addition was made to the measured stretch because the latter, in all cases, was well above the specified minimum total elongation.

The use of two extensometers in connection with the yield-point test was necessary for the reason that a sensitive instrument required to indicate small elongations would be injured from blows caused by the rupture of the specimen. For measuring the stretch from the initial load to the yield point a 10-in. extensometer was used and then removed. This extensometer was equipped with an Ames dial registering increments of 0.001 in. of stretch. The other or sturdier instrument, which was attached to the specimen at the same time, had a dial graduated to hundredths of an inch, and this instrument remained on the specimen until rupture, so that the operator could observe the indicator at the instant of rupture.

The average results of 26 274 tests, or those for which the yield point was measured, are given in Table 8. They cover 958 melts of wire.

TABLE 8.—RESULTS OF YIELD-POINT TESTS OF CABLE WIRE

	Minimum	Average	Maximum
Tensile strength, in pounds per square inch	220 000	234 000	259 000
Yield point, in pounds per square inch	150 000	184 000	202 000
Percentage elongation, in 10 in.	.....	6.0	.....
Diameter, in inches	.....	0.1965	.....

Rejections amounted to 0.7% of the total wire used. Of this quantity, 74% represents material having physical and chemical properties outside the specified limits; 13%, surface inperfections; and the remainder includes material which showed such undesirable qualities as "off gauge", kinks or waves, and brittleness.

The modulus of elasticity of the wire was determined for use in connection with the study of testing apparatus and procedure and for other purposes, outside the scope of this paper. The modulus tests were made with 300-in. gauge lengths on specimens, each from a different melt. In the preliminary investigations, eight tests were made, the results of which indicated an average modulus of 28 000 000 lb per sq in. Additional tests, ten in number, made during the manufacturing period, revealed an average value of 28 700 000 lb per sq in. Although the results of the latter tests would seem to indicate that this wire has a modulus of elasticity somewhat higher than was assumed when establishing the routine testing procedure used in performing the yield-point test, no change was made in the procedure because the calculated stretch for the initial load remained the same for all practical

Stress in Thousand Pounds per Square Inch

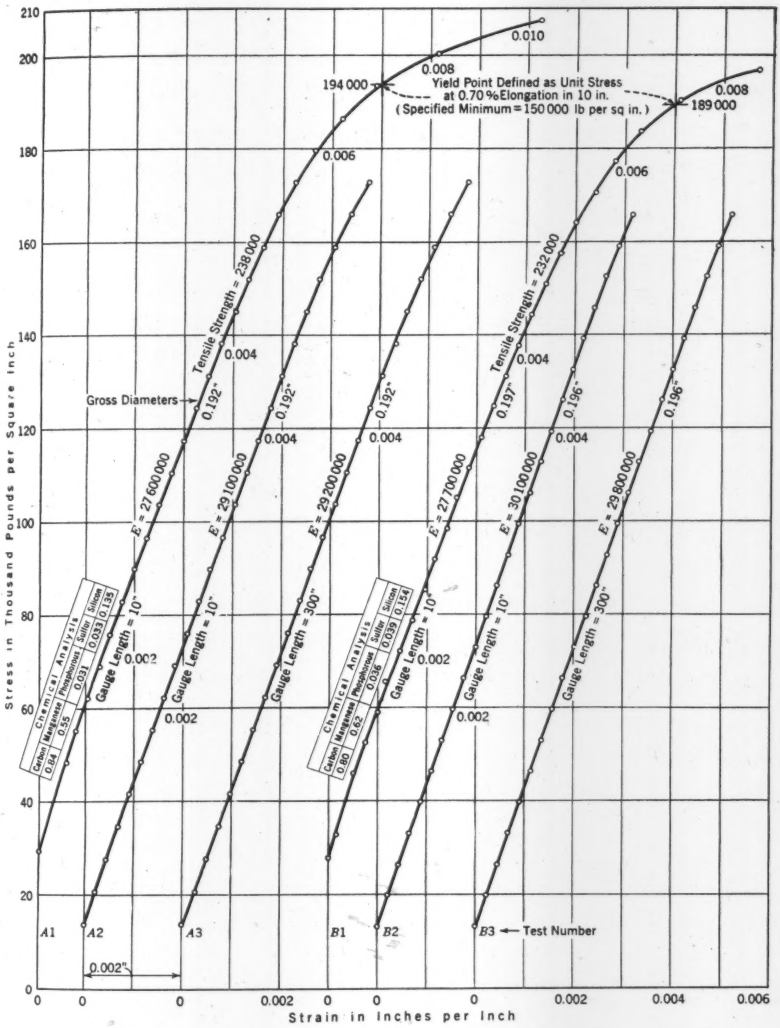


FIG. 17.—STRESS-STRAIN CHARACTERISTICS OF CABLE WIRE.

purposes. The results of some of the tests have been plotted in Fig. 17. The gross diameters indicated on each curve include the thickness of the galvanizing. The tests in Group *A* (*A*1, *A*2, and *A*3) were made on one length of wire. Those in Group *B* (*B*1, *B*2, and *B*3) likewise were made on one length of wire, but from a different melt. The test results indicated by

Curves $A1$ and $B1$ were obtained with acceptance test apparatus on specimens (10-in. gauge length) from respective lengths. The test results indicated by Curves $A2$ and $B2$, were obtained with a 10-in. acceptance-test extensometer attached near the mid-length (300-in. gauge length) of specimens for Tests $A3$ and $B3$, respectively, and the two sets of tests were conducted simultaneously.

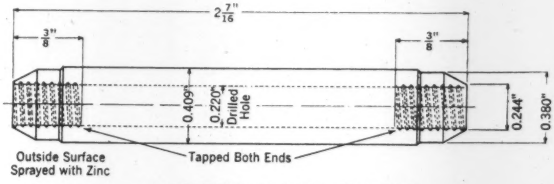
TABLE 9.—PHYSICAL PROPERTIES THROUGHOUT COILS OF CABLE WIRE
(Specimens Taken Every 100 Feet)

Coil No. 1					Coil No. 2				
	Yield Point		Tensile Strength			Yield Point		Tensile Strength	
Diameter, in inches	Pounds	Thousand pounds per square inch	Pounds	Thousand pounds per square inch	Diameter, in inches	Pounds	Thousand pounds per square inch	Pounds	Thousand pounds per square inch
0.197	5 430	178	7 020	230	0.196	5 580	185	7 440	247
0.200	5 490	175	7 060	225	0.196	5 640	187	7 470	248
0.197	5 560	182	7 180	236	0.196	5 600	186	7 500	249
0.198	5 520	179	7 180	233	0.197	5 180	170	7 550	248
0.198	5 540	180	7 170	233	0.196	5 700	189	7 450	247
0.198	5 580	181	7 190	234	0.197	5 620	184	7 530	247
0.199	5 460	176	7 210	232	0.197	5 700	187	7 530	247
0.197	5 550	182	7 160	235	0.196	5 700	189	7 560	251
0.197	5 530	181	7 180	236	0.196	5 420	180	7 540	250
0.197	5 620	184	7 190	236	0.196	5 600	186	7 540	250
0.198	5 530	180	7 170	233	0.196	5 520	183	7 520	249
0.198	5 590	182	7 160	233	0.196	5 550	184	7 530	250
0.197	5 550	182	7 160	235	0.196	5 530	183	7 450	247
0.198	5 560	181	7 180	233	0.196	5 540	184	7 490	248
0.197	5 560	182	7 190	236	0.196	5 410	179	7 470	248
0.197	5 560	182	7 180	236	0.196	5 060	168	7 570	251
0.198	5 500	179	7 160	233	0.196	5 560	184	7 600	252
0.198	5 580	181	7 180	233	0.196	5 570	185	7 540	250
0.198	5 600	182	7 180	233	0.196	5 500	182	7 560	251
0.196	5 560	184	7 190	238	0.196	5 540	184	7 450	247
0.198	5 590	182	7 220	234	0.196	5 600	186	7 540	250
0.197	5 540	182	7 140	234	0.196	5 630	187	7 550	250
0.198	5 560	181	7 180	233	0.196	5 590	185	7 540	250
0.197	5 550	182	7 180	236	0.196	5 620	186	7 550	250
0.197	5 520	181	7 200	236	0.196	5 560	184	7 500	249
0.196	5 580	185	7 190	238	0.196	5 560	184	7 500	249
0.196	5 540	184	7 180	238	0.196	5 520	183	7 500	249
0.196	5 540	184	7 160	237	0.196	5 580	185	7 520	249
0.196	5 530	183	7 190	238	0.196	5 570	185	7 520	249
0.196	5 580	185	7 210	239	0.196	5 560	184	7 480	248
0.196	5 610	186	7 240	240	0.196	5 260	174	7 560	251
0.196	5 530	183	7 220	239	0.196	5 680	188	7 520	249
0.197	5 560	182	7 200	236	0.196	5 580	185	7 520	249
0.196	5 550	184	7 160	237	0.196	5 580	185	7 520	249
0.197	5 510	181	7 190	236	0.196	5 160	171	7 520	249
0.197	5 550	182	7 120	234	0.196	5 460	181	7 490	248
					0.196	5 560	184	7 500	249
					0.196	5 540	184	7 520	249
AVERAGE									
0.197	5 550	182	7 170	235	0.196	5 540	183	7 520	249

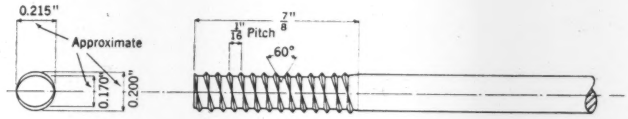
The degree of uniformity in the physical properties throughout the length of a coil was investigated by testing two coils of wire from different melts. A specimen was taken every 10 ft and tested for yield point and tensile strength. The results of tests on specimens from the 100-ft points are recorded in Table 9. These tests are listed in the order the samples were cut from one end of the coil, and they include the extreme values found. The averages in Table 9 are the same as those found for all the tests per

coil. It should be noted that in studying the values in this table consideration must be given to the fact that there are differences, caused by nonuniformity in thickness of zinc coating, in the gross diameter of the specimens, and, for this reason, the load values form a more reliable indication of the uniformity of the material.

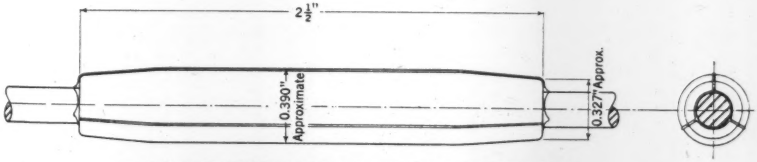
Splices.—The individual lengths or coils of wire were spliced together at the wire plant during the reeling operation. Three steps constituted the splicing process: First, preparing the ends of the wire; second, assembling



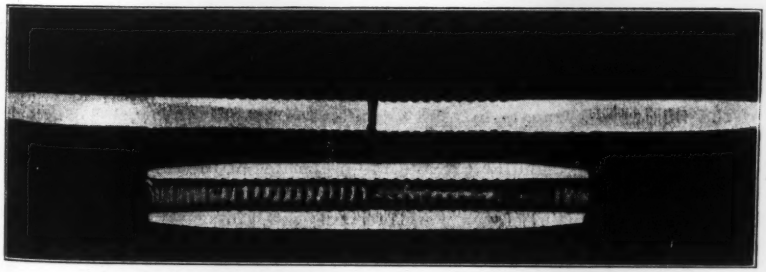
(*a*) SLEEVE BEFORE PRESSING



(*b*) PRESSED END OF CABLE WIRE



(*c*) SLEEVE PRESSED ON WITH THREE-PART DIE



(*d*) SECTION OF CABLE WIRE SPLICE WITH WIRES REMOVED FROM SLEEVE

FIG. 18.—DETAILS OF CABLE WIRE SPLICE.

them in the galvanized sleeve; and, finally, making the splice. Although this procedure is fundamental, the details underlying it differed from previous practice of making the splice by the method of drawing threaded ends together with a threaded sleeve.

In explanation of the method used, it should be pointed out that the result desired in the splice was one that would develop at least 95% of the specified minimum tensile strength of the wire and prevent failure due to untwisting. Accordingly, the wire ends were corrugated by pressure to give the necessary frictional resistance to the pull and, in addition, made elliptical to afford resistance against turning. The splice was completed by pressing a galvanized sleeve on the deformed wire ends. In the latter operation, the pressure was applied to the sleeve in two applications through a three-part die, the splice being turned through 60° after the first application. Fig. 18 shows details of the splice and Fig. 18(*d*) shows a section of a splice with parts dis-assembled.

There were 11 985 splice tests made. Slightly more than 4% failed to show the specified minimum value for strength, failure occurring in practically all cases by the wire pulling out of the sleeve. Based on the average ultimate strength and diameter obtained for the yield-point tests in Table 8 and the average load of 6 902 lb sustained by the splices, the average efficiency amounts to 98 per cent.

(5) Suspender Rope

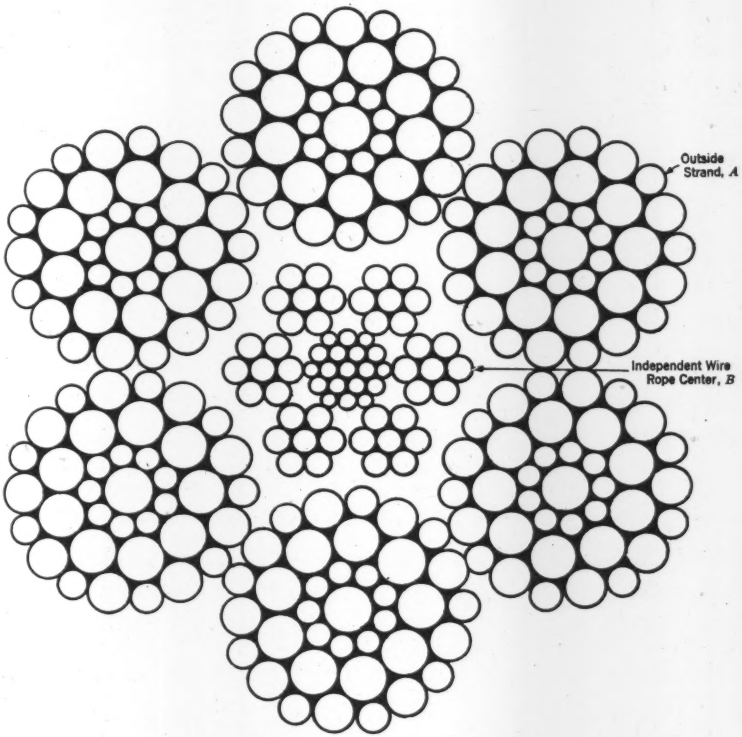
Suspender ropes are the structural elements by which the floor system is hung from the cables. They are composed of galvanized wires of various sizes which have been arranged into six strands around an independent wire rope center (see Fig. 19). The diameter of the suspender rope is about $2\frac{1}{8}$ in. The lay of the rope is approximately 25 in. About 170 200 lin ft of rope are in place in the structure.

Before the suspender ropes were placed in position they first served as the supporting members of the footbridges used in the erection of the cables. For this reason, the rope was shipped from the manufacturing plant in maximum and minimum lengths of about 3 500 and 760 ft, respectively, and eventually made into suspender ropes at the bridge site.

An interesting feature instituted in the production of this rope was an operation called "pre-stressing". Its use was intended to reduce the inelastic stretch, or that stretch which occurs from the failure, during manufacture, of the wire rope elements to work themselves into close contact. Although not required by the specifications, pre-stressing is, nevertheless, closely associated with the manufacture and the properties of the rope. The account that follows, therefore, after indicating the principal manufacturing steps, outlines briefly this operation, and records the properties of the finished product.

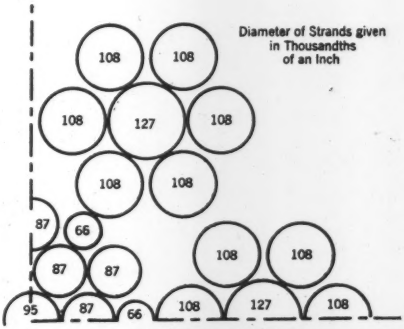
Manufacture.—The steps in the manufacture of the individual wires duplicated those described under the heading "Cable Wire".

Fabrication of the rope consisted essentially of three operations; First, making the strands from the individual wires; second, combining strands



(a) CROSS-SECTION OF $2\frac{7}{8}$" SUSPENDER ROPE

(b) ENLARGED $\frac{1}{4}$ SECTION OF OUTSIDE STRAND *A* SHOWING ARRANGEMENT AND SIZES OF WIRES



(c) ENLARGED $\frac{1}{4}$ SECTION OF INDEPENDENT WIRE ROPE CENTER *B* SHOWING ARRANGEMENT AND SIZES OF WIRES

FIG. 19.—CROSS-SECTION DIMENSIONS OF SUSPENDER ROPE.

to form the independent wire rope center; and, finally, closing the outside strands about the rope center. Strands were made in lengths sufficient to produce about 7 200 lin ft of rope. To make strands of sufficient length, the individual wires were spliced by means of brazing, the customary practice.

Pre-Stressing Operation.—To subject the suspender rope to stress prior to its use as erection material was considered necessary by the manufacturer, in his scheme of erection. A description of the plans is, of course, outside the scope of this paper; but for the purpose of explaining the pre-stressing procedure there is pointed out a common characteristic of wire rope and the change to be effected in this product by the operation.

From the nature of its construction, the fact is evident that under load wire rope continues to elongate until the wires adjust themselves into a position at which the load can be resisted. As already referred to, it was desired to seat the elements of the rope among themselves through some system of loading so that the member would lose its inelastic stretch.

The apparatus used for the pre-stressing operation consisted of the following principal parts: (1) A length of railroad track for a car, upon which was mounted, horizontally, a sheave 8 ft in diameter; (2) at one end of the track, an hydraulic testing machine; and (3) at the other end of the track, two jacks, with appliances for holding the rope ends, one on either side of, and parallel to, the track.

The procedure was essentially as follows: One-half a manufactured length of rope was placed in position by attaching the ends to the jacks and passing a bight around the sheave. One end was pulled by a jack until load was registered by the testing machine to which the sheave-car was attached. The testing machine was then brought into play for raising the load to 400 000 lb and for sustaining this load for 10 hours. At the end of this time, the tension (200 000 lb) in the rope was released. Following this step in the operation, the rope was measured and cut to lengths required in the footbridge construction. The measurements for length were made while the tension was 80 000 lb, which was equalized in the two parts of the rope by manipulating jacks and testing machine.

The influence of the pre-stressing operation on the elastic properties of this rope may be observed from comparison of the characteristics of non-stressed and pre-stressed rope shown in Fig. 20. The theoretical cross-sectional area of the suspender rope is 4.0474 sq in. Points *C* and *D*, in Fig. 20, are the specified maximum elongations of 0.3 in. and 0.5 in. in a gauge length of 100 in., between loads of 5 000 and 205 000 lb per sq in., for pre-stressed rope and for rope not previously stressed, respectively. Test No. 1 was made of rope not previously stressed; the other tests were made on pre-stressed rope.

Tests and Physical Properties.—Tests were performed on the single wire lengths and on the rope itself. The former consisted particularly of the Preece test, although tensile tests were made on each coil of galvanized wire to insure attainment of a uniform raw material. The full-sized tests comprised two specimens cut from each manufactured length, one simulating

a condition in the structure for obtaining the tensile strength as the rope lay over a sheave, producing a radius of 21 in. in the center line of the rope; and the other for the purpose of gauging the degree of stretch and observing the ultimate strength in a single-part test.

Sockets from among those made for use in the structure were attached to the ends of the rope specimens to aid in the testing.

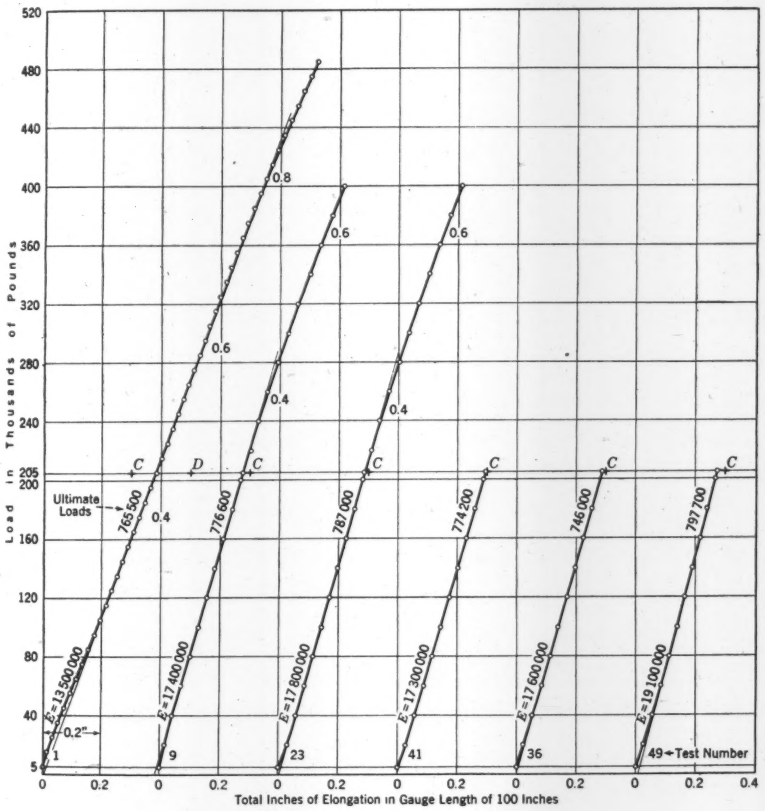


FIG. 20.—STRESS-STRAIN CHARACTERISTICS OF SUSPENDER ROPE.

For the full-sized tests, the specifications required that: (*a*) A rope tested over a sheave should develop a total strength in both parts of not less that 1 200 000 lb; (*b*) a rope tested straight should show, by strain measurement in a gauge length of 100 in., a stretch of not more than $\frac{1}{2}$ in. from an initial load of 5 000 lb to one of 205 000 lb, for rope not previously loaded, nor more than 0.3 in. for rope after it has been subjected to repeated loading

up to 205 000 lb; and (*c*) the stretch of any rope should not vary more than 10% from the average stretch of all ropes tested. The results of the single and two-part tests are presented in Table 10.

TABLE 10.—RESULTS OF TESTS ON PRE-STRESSED SUSPENDER ROPE 2⅞ INCHES IN DIAMETER

Reel No.	Single Part Tests			Double part tests, ultimate strength, in pounds
	Ultimate strength, in pounds	Total elongation, in 100 in.,* in inches	Modulus of elasticity, in pounds per square inch	
1	745 400	0.2940	18 600 000	1 328 000†
2–3	788 000	0.2760	18 900 000	1 369 200
4–5	770 000	0.2825	17 600 000	1 337 000
6–7	794 800	0.2995	17 300 000	1 361 400
8–9	776 600	0.2760	17 400 000	1 363 700
10–11	788 400	0.2960	17 900 000	1 359 500
12–13	777 000	0.2995	17 300 000	1 363 500
14–15	775 400	0.2875	18 000 000	1 362 700
16–17	788 600	0.2910	17 800 000	1 365 000
18–19	776 000	0.2900	18 200 000	1 312 700
20–21	772 800	0.2850	18 200 000	1 306 700
22–23	787 000	0.2850	17 500 000	1 380 800
24–25	780 000	0.2775	18 600 000	1 387 200
26–27	791 000	0.2830	18 200 000	1 304 000
28–29	786 200	0.2810	17 800 000	1 380 200
30–31	791 400	0.2880	17 400 000	1 373 300
32–33	774 200	0.2920	17 600 000	1 376 600
34–35	780 800	0.2835	17 900 000	1 356 200
36	746 000	0.2880	17 700 000	‡
37–38	766 000	0.2825	19 000 000	1 291 600
39–40	790 000	0.2895	17 700 000	1 367 200
41–42	774 200	0.2955	17 300 000	1 323 000
43–44	733 400	0.2835	17 600 000	1 242 800
45–46	777 100	0.2785	18 700 000	1 280 800
47–48	784 400	0.2850	18 500 000	1 244 200
49–50	797 700	0.2745	19 100 000	1 363 000
51–52	788 200	0.2845	18 100 000	1 285 400
53–54	798 200	0.2890	18 000 000	1 271 400
Average	778 500	0.2863	18 000 000	1 335 700

* Between 5 000 and 205 000 lb.
† Ultimate strength of rope in double part test not previously stressed.
‡ No double part test made.

(6) OTHER MATERIALS

In addition to the heat-treated eye-bars, structural and cast steels, cable wire, and suspender rope incorporated in the main steel structure and described heretofore, other kinds of metallic materials are: Heat-treated, forged, and rolled carbon steel; annealed forged carbon steel; rolled manganese-bronze and cast phosphor-bronze; and wrapping wire.

In the description that follows, the principal features concerning dimensions, quantity, manufacture, and properties are given consideration in regard to these materials.

Heat-Treated Steel Pins.—The heat-treated steel pins that connect the eye-bar chains to the girders embedded in the anchorages are 2 ft. 5.5 in. in length; there are two sizes, those in the New York anchorage being 10 in. in diameter and those in the New Jersey anchorage, 11½ in. in diameter. In all, 100 pins were required for the two anchorages.

Briefly, the manufacturing operations in producing the pins consisted of forging, heat-treating, and machining. These operations were conducted in accordance with the best modern practice.

The properties of the pin material as determined by the ladle analyses and the acceptance tests are given in Table 11, as are also the requirements that the material had to meet.

TABLE 11.—SPECIFIED AND AVERAGE PROPERTIES OF HEAT-TREATED PINS AND BOLTS*

Part of structure	Number of tests	PHYSICAL				CHEMICAL			
		Yield point, in pounds per square inch	Tensile strength, in pounds per square inch	Percentage elongation, in 2 inches	Percentage reduction of area	Carbon	Manganese	Phosphorus	Silicon
Specified requirements..	....	60 000†	95 000†	21.0†	45.0†	.....	.....	0.04‡	0.05‡
Cable band bolts.....	74	72 100	103 200	24	61	0.51	0.60	0.018	0.034
Eye-bar pins‡.....	9	63 600	101 300	25	56	0.51	0.65	0.023	0.029
Average.....	....	71 200	103 000	24	60	0.51	0.62	0.019	0.032

* Axis of specimen located midway between center and surface of member. † Minimum. ‡ Maximum.

The acceptance tests consisted of one tension and one bend test for each melt represented in each heat-treated lot. The specimens were of the usual diameter (0.5 in.) and were selected from points midway between the center and surface of the forgings. The bend specimen withstood a cold bend through 180° around a mandrel $\frac{1}{2}$ in. in diameter without showing signs of rupture on the outside of the bent part.

Heat-Treated Cable Band Bolts.—The finished bolt is $2\frac{3}{8}$ in. in diameter and 2 ft 3 in. in length under the head. A total of 3 400 bolts was required.

The manufacturing operations followed in sequence the steps of rolling, forging the head, heat-treatment, and machining. The billets were hot-rolled into bars of a nominal diameter of $2\frac{1}{2}$ in. The latter were cut into pieces long enough to permit forging the head. The resultant blanks were heated in a continuous furnace, quenched in water, and subsequently were drawn in a similar furnace.

The chemical and physical requirements that the heat-treated bolt material had to meet were the same as those specified for the heat-treated pin steel. (See Table 11.)

Specimen tests, one for tension and one for bend, were made for each heat-treated lot of fifty bolts. The bend specimens withstood a 180° flat bend. The average results of the specimen tension tests and the average results of the ladle analyses are shown in Table 11. In addition to the specimen tests, an occasional full-sized bolt was tested in tension. These full-sized tests (ten in number), indicated a range from 66 000 to 85 000 lb sq in. for the yield point, and from 109 000 to 127 900 lb per sq in. for the tensile strength, the average being 73 920 and 116 900 lb per sq in., respectively. Failure in each case occurred in the threads.

The stress-strain characteristics shown graphically in Fig. 21 are typical of the material. The five melts of steel from which the required number of

bolts were made, are there represented; one of them (Melt *E*, Fig. 21) is of Mayari or chrome-nickel steel. Specimens were cut from a point midway between the center and surface of the heat-treated, unfinished bolt, and were machined to a diameter of 0.505 in. The dotted extensions to the curves in Fig. 21 indicate excessive stretch (not less than 0.009 in. per in.) that occurred before the next load increment had been applied. The yield point

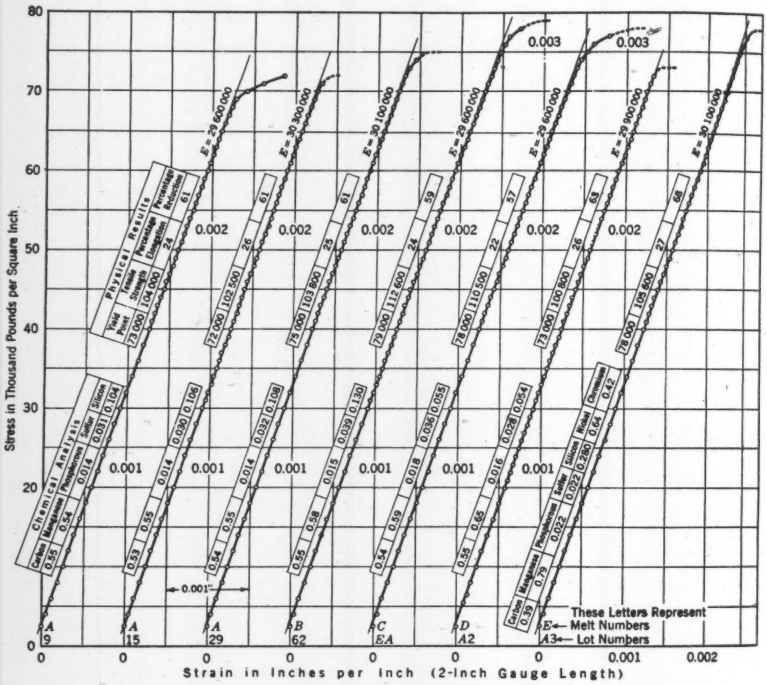


FIG. 21.—STRESS-STRAIN CHARACTERISTICS OF HEAT-TREATED CABLE BAND BOLTS.

(the drop of the beam), was considered when plotting the broken line. Samples for chemical analysis were milled from the broken tension specimens.

Annealed Steel Pins, Rollers, and Rockers.—The number of annealed forged pins in the eye-bar chains in the anchorages is 992; of rollers under the tower cable saddles, 328; of rockers under the anchorage cable saddles, 120; and of miscellaneous pins in the floor system, 14. The anchorage eye-bar pins are 10 in. in diameter and from 12 in. to 2 ft 5½ in. in length; the rollers, 8 in. in diameter and 8 ft 6 in. in length; the rockers, 12 in. in diameter, but with two parallel sides giving a 7-in. width, and 7 ft in length; and the floor system pins are of various diameters and lengths. The manufacturing operations on these members were similar to those outlined in this paper for the heat-treated pins, except that they were annealed.

This material was manufactured in accordance with the Specifications for Class F Annealed Carbon Steel Forgings of the American Society for Testing Materials (Serial Designation A18-27). The properties of the annealed material are given in Table 12. The stress-strain curves of three specimen tests representing rockers are plotted in Fig. 3.

TABLE 12.—SPECIFIED AND AVERAGE PROPERTIES OF ANNEALED FORGED CARBON STEEL*

Part of structure	Number of tests	PHYSICAL				CHEMICAL			
		Yield point, in pounds per square inch	Tensile strength, in pounds per square inch	Percentage elongation, in 2 inches	Percentage reduction of area	Carbon	Manganese	Phosphorus	Silicon
Specified requirements	....	$\frac{\text{Tensile strength}}{2}$	75 000†	$\frac{1\,725\,000}{\text{Tensile strength}}$	$\frac{2\,640\,000}{\text{Tensile strength}}$	.....	0.40 to 0.80	0.05‡	0.05‡
Eye-bar pins	33	40 600	78 000	27	44	0.38	0.53	0.024	0.034
Anchorage saddle rockers	6	43 800	82 500	25	41	0.39	0.66	0.014	0.035
Tower saddle rollers	26	44 300	78 200	28	43	0.39	0.59	0.018	0.034
Other pins	3	48 200	86 100	25	42	0.44	0.63	0.026	0.035
Average	....	42 700	78 800	27	43	0.39	0.59	0.020	0.034

* Axis of specimen located midway between center and surface of member. † Minimum. ‡ Maximum.

Bronze.—Rolled manganese-bronze wearing plates are used at expansion points in the floor system and cast phosphor-bronze is used for bushing pinholes. A total of about 6 450 lb of these materials is in service in the main structure.

The quality of the materials was investigated by tension and compression tests, both kinds being made on the manganese-bronze, and only compression tests on the phosphor-bronze. The rolled material revealed, in two tension tests, an average yield point of 77 250 and tensile strength of 112 000 lb per sq in., a total elongation of 18% in 2 in., and a reduction in area of 22 per cent. For these properties the specifications required, respectively, 55 000 and 100 000 lb per sq in. minimum, and 15% minimum in the deformations. The specified properties and average results of the compression tests and chemical analysis of this material are given in Table 13.

Wrapping Wire.—The material with which the cables are wrapped is galvanized wire about 0.148 in. in diameter. Approximately 409 tons of this wire were required.

The steel was made by the basic open-hearth process and is of the following average analysis: Carbon, 0.10; manganese, 0.50; phosphorus, 0.020; and sulfur, 0.030. The material was manufactured essentially in the same manner as the cable wire.

The quality of the wire was investigated by the tension test and the quality of the galvanizing by the Preece test. The tensile strength and total elongation were measured, although the specifications required only the determination of elongation in 10 in., the minimum value for which had to be

10 per cent. The average tensile strength of the wire is about 68 000 lb per sq in., based on the gross cross-sectional area, which includes the zinc coat; the average elongation proved to be about 16 per cent. One end of each coil was tested for tensile strength and elongation, and 5% of the coils were tested for quality of galvanizing.

TABLE 13.—SPECIFIED PROPERTIES AND AVERAGE RESULTS OF COMPRESSION TESTS ON ROLLED MANAGANESE-BRONZE AND CAST PHOSPHOR-BRONZE

Item No.	Description	Number of tests	Physical Properties		Chemical Properties								
			Load at permanent set of 0.001 in., in thousands of pounds	Permanent set at load of 100 000 lb per sq in., in inches	Copper	Zinc	Manganese	Phosphorus	Iron	Aluminum	Lead	Tin	Remainder
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
Rolled Manganese-Bronze; Specimen, 0.75 Inch by 1.33 Inches by 1.00 Inch													
1	Specified requirement	....	50.0*	0.05†	62 to 68	23 to 29	....	....	....	....	....	....	.....
2	Average	2	58.7	0.024	67.56	20.79	3.72	0.025	1.18	6.06	0.66	....	.....
Cast Phosphor-Bronze; Specimen, 1.128 Inches in Diameter by 1.00 Inch in Length													
3	Specified requirement	....	19.0*	0.25†	81.5*	.....	....	1.0†	....	....	....	17.0†	0.5†
4	Average	16	24.6	0.12	85.73	.....	....	0.64	....	....	....	13.55	0.08

* Minimum. † Maximum.

The coils (each of which weighed about 175 lb) were spliced to form long lengths for shipment to the bridge site. The splice was made by butt-welding and had an efficiency of about 98 per cent. In most cases, the weld-test specimen ruptured a short distance from the junction of the two wires.

TABLE 14.—MATERIALS IN THE MAIN STEEL STRUCTURE, IN TONS

Item No. (1)	Material (2)	Anchorages (3)	Towers (4)	Cables (5)	Suspended structure (6)	Total (7)
	Heat-Treated Steel:					
1	Eye-bars	4 481	.....	.....	.....	4 594
2	Steel pins	36	.....	.....	.....	
3	Steel bolts	.....	.....	77	.....	
	Structural Steel:					
4	Carbon	1 825*	18 254	36	8 869	28 984
5	Silicon	.....	23 587	.....	8 132	31 719
6	Cast steel	397†	1 450§	728	68	2 643
7	Annealed forged steel	342‡	238¶	.....	3‖	583
8	Cable wire	.....	.....	28 308	.....	28 308
9	Wrapping wire	.....	.....	409	.....	409
10	Suspender rope	.....	.....	1 234**	.....	1 234
11	Hand ropes	.....	.....	43††	.....	43
12	Reinforcement (bulb-beams and tie-rods in roadway)	.....	.....	.....	2 341	2 341
13	Bronze	.....	.....	.....	3	3
	Miscellaneous:					
14	Railings	.....	.....	.....	636	636
15	Conduits, light standards, etc.	.....	.....	.....	261	261
16	Total	7 081	43 529	30 835	20 313	101 758

* Excluding anchorage floor system. † Anchorage saddles and strand shoes. ‡ Pins and rockers
§ Tower saddles. ¶ Rollers. ‖ Pins. ** 14.5 lb per lin ft. †† 2.2 lb per lin ft.

(7) Distribution of Materials in Main Divisions of Structure

Approximately 100 000 tons of metallic materials were required in the construction of the main bridge structure not including the approaches. The quantity of each kind of material in each of the four principal divisions of the main steel superstructure is given in Table 14. The quantities given do not include the floor system in the anchorages, or the concrete reinforcing rods.

Summary

The writer wishes to emphasize again the fact that the materials of the structure and the methods of manufacture are in no way experimental, but in every case are representative of the best modern practice. In dealing with the subject-matter of this paper he has intentionally treated briefly, or has omitted entirely, a detailed description of such phases of the shop work as are met with in every-day common practice, even though such operations are of interest to those unfamiliar with the methods of manufacture. Feeling that such descriptions may be found elsewhere, and are not properly a part of this paper, he has rather attempted to indicate the conformity of the work with the most modern manufacturing methods and also with the specifications, and to point out such modifications in requirements of materials or in methods as are applicable to this work. The program of tests and the characteristics of materials incorporated in the structure are treated in some detail because it is believed that, in view of the magnitude of the operation and the care taken in following a comprehensive testing program, this phase of the subject will be of special value to the Engineering Profession.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 1825

GEORGE WASHINGTON BRIDGE: APPROACHES AND HIGHWAY CONNECTIONS

By J. C. Evans,[1] Esq.

Synopsis

The approaches to the George Washington Bridge, including an outline of the steps in the development of the plans from their simplified beginnings, are described in this paper. It contains a discussion of the construction of the New York approach in stages to conform to traffic requirements and financial limitations.

Radically different conditions were met at the two ends of the bridge. In Manhattan, the approaches are in a highly congested area, while in New Jersey they are in a suburban territory. The history of the development of the plans for these approaches indicates the great amount of time and effort required to arrive at satisfactory solutions of the problems. The close co-operation which was sought and obtained between the Port Authority, as the constructing body, and the Municipal, County, and State Governments, was an important factor in the work.

Location of Bridge to Meet Traffic Needs

The location of the bridge at Fort Washington Park, between 178th Street and 179th Street, in Manhattan, and at Fort Lee, N. J., fills a definite and pressing traffic need of the Metropolitan Area. The bridge is ideally situated for traffic from the south and southwest bound to and from points in New England. At this location such traffic may avoid the congested area of Lower Manhattan. The bridge is also well situated to serve the needs of a rapidly growing suburban area of Northern New Jersey. In recent years, Upper Manhattan and the contiguous areas have been growing in population at a rapid rate, and this territory also demands facilities for access to the New Jersey area.

At the bridge site, Manhattan Island is roughly $1\frac{1}{4}$ miles wide between the Hudson and the Harlem Rivers. Riverside Drive skirts the Hudson

[1] Terminal Engr., The Port of New York Authority, New York, N. Y.

River and Fort Washington Park and bounds this territory on the west, while the Harlem River Speedway skirts it on the east. Between these prominent highways are several important north and south arteries, the most important of which are, from west to east, Fort Washington Avenue, Broadway, St. Nicholas Avenue, and Amsterdam Avenue. Almost directly east of the site of the George Washington Bridge, the Harlem River is crossed at 181st Street by the Washington Bridge which connects with University Avenue in The Bronx and thence with all other principal arteries of that area.

In New Jersey at the time the bridge location was chosen no principal modern traffic artery passed through Fort Lee. The Borough itself is a comparatively small community of 9 000 population. It had three fairly good highways—Lemoine Avenue leading north in the direction of Englewood, Palisade Avenue leading south, and Fort Lee Turnpike leading west to Hackensack and the Paterson and Passaic area—but in the sense of the modern highway, it had none. The fine modern highways that, when completed, will focus upon the bridge and carry its traffic by direct routes to all parts of the New Jersey area, are due to the progressive and co-operative attitude of the New Jersey State Highway Commission that has provided the means by which the full utility of the bridge can be realized in the New Jersey territory.

Approaches as Major Elements of the Bridge Project

The choice of the exact location of the structure was determined largely by construction considerations, such as span and foundation conditions. The legislation directing the Port Authority to construct the bridge specified a location between 170th Street and 185th Street, in Manhattan, and a point approximately opposite thereto in the Borough of Fort Lee in New Jersey. Within these limits three sites were studied. The final choice was dictated largely by the fact that studies indicated the location between 178th Street and 179th Street as the most desirable with respect to approaches, grades, and street connections. Fortunately, the studies also indicated the location chosen to be the most economical from a construction standpoint. In New Jersey, the location was such as to permit the development of the approach with its greater space requirements for toll collection in a part of the territory north of the main business center of Fort Lee, thus utilizing less expensive property and offering the minimum of interference to local development, and, at the same time, giving maximum accessibility to the business district and to the principal traffic arteries converging in this locality.

The main bridge structure fulfills its purpose only to the extent that the approaches are able to distribute the traffic adequately. This is especially true in the case of an approach in Upper Manhattan where the traffic arteries were already carrying a high percentage of their estimated capacities at the time of the opening of the George Washington Bridge. The actual bridge roadway opened to traffic was limited accordingly, under initial conditions, to four lanes—the maximum amount which the municipal authorities believed could be handled adequately by the connections.

The old conception of a bridge approach as a simple ramp from the main structure to the ground surface, terminating in a plaza, has required considerable revision under such circumstances. There have been diverse views as to what constitute approaches, and the Port Authority, through its extended negotiations with the municipalities, has adopted a broad viewpoint to the effect that the approaches should not merely embrace the ramps leading from the bridge proper to wherever they may strike the ground, but that they should also include adequate connections between the ramps and such existing through arteries as will assure the proper flow and distribution of the bridge traffic to such arteries. It was recognized that the municipalities should not be burdened with the cost of such necessary connections when they are to carry predominantly through traffic over the bridge. On the other hand, the construction by the Port Authority of adequate connections to assure the unhindered maximum flow of traffic to and from the bridge is unquestionably in conformity with sound business policy and in protection of the bond-holder's interests. Ramps or tunnels communicating directly with arteries of adequate capacity have thus been incorporated in the approach plans.

In New Jersey, the development of the approach plans was such as to make it extremely difficult to say where the bridge approach should end and the State highway system begin, the bridge itself becoming in fact an extension of a high-speed ultra-modern State highway system.

The relation between the monies expended in developing the approach facilities and the cost of construction of the main structure gives an indication of the major importance of the approach elements. The cost of the approaches accounts for about 21% of the total sum expended to date (1933) on construction by the Port Authority. Including real estate, engineering, administration, and expenses of financing, the approaches account for approximately 37% of the expenditures of the Port Authority on the project. If, in addition, the cost of the new roadways constructed in New Jersey by the State Highway Commission directly for the benefit of bridge traffic is considered, the total cost of approach and approach highway connections is well over 40% of the total cost of the facility.

Approval of Plans by Governors and Municipalities

The statutes authorizing the construction of the bridge provided that the plans for the approaches should be subject to the approval of the Governors of the two States and of the respective municipalities in which the facilities were to be located. Such an arrangement is only proper and the Port Authority early sought the counsel and approval of the officials of the City of New York and the Borough of Fort Lee, as well as the approval of all other county and State highway bodies interested in the bridge project.

The approach plans for both sides of the river, as they have been adopted finally, embody largely the ideas of the officials who have acted with representatives of the Port Authority on joint committees appointed to develop the plans. As presented in the early studies, the approaches underwent

repeated modifications and improvements; perhaps no better illustration can be found of the advantage of the continuous development of a problem of such intricate character over a period of a number of months, requiring as it does complete co-operation between different interests. It would have been impossible to secure as satisfactory results in this work had it been necessary to prepare complete plans before contracting for any work on the structure.

Development of the New York Approach Plan

One of the early proposals for the New York approach provided for its construction in two stages. In the initial stage the approaches were to occupy only the blocks between 178th and 179th Streets, from Haven Avenue to Fort Washington Avenue, with a plaza between these two streets from Fort Washington Avenue to Broadway, as shown on Fig. 1. It was believed at that time that the initial stage as proposed would make adequate provision for the bridge traffic for a period of five to eight years following completion of the initial stage in 1932. During this period bridge traffic to and east of Broadway could be accommodated by widening West 178th and West 179th Streets, between Fort Washington Avenue and Broadway, with crossing of bridge traffic and street traffic at grade on Fort Washington Avenue. It was intended that traffic to and from Riverside Drive would utilize 178th and 179th Streets, and existing connecting streets west of Fort Washington Avenue.

The ultimate stage as then planned is shown on Fig. 2, and indicates a continuation of the boulevard treatment from Broadway to Amsterdam Avenue, connecting the approaches to the George Weshington Bridge with the approaches to the Washington Bridge over the Harlem River. The boulevard provided for the crossing of intersecting streets at grade, except Broadway, which thoroughfare was to be depressed under the boulevard, and a traffic circle established in order to avoid interference of bridge traffic by or with through Broadway traffic.

The entire question of location and approach plans was submitted to the City Officials for consideration as soon as the arrangements for financing were completed. The mutual interests of the City and the Port Authority in the bridge project were considered by a Joint Conference Committee of City and Port Authority representatives. At a meeting of this Committee in January, 1927, an Engineering Sub-Committee was appointed, consisting of the Chief Engineers of the Board of Estimate and Apportionment, the Borough of Manhattan, the Board of Transportation, the Department of Plant and Structures, the Dock Department, and representatives of the Park Department and the Department of Finance, for the City, and the Chief Engineer and two Consulting Engineers, representing the Port Authority. This Committee had before it decision as to the site for the structure in Fort Washington Park and plans for the approaches.

At first, it was the feeling of the municipal representatives on the Committee that the question of approach plans and approval by the City, of the bridge site, were so bound together that decision on one could not be made

without, at the same time, reaching decision on the other. However, in March, 1927, it became apparent that a comprehensive plan of street connections and arterial routes should be developed without regard to terms and conditions as between the Port Authority and the City. It was likely that the work would involve considerable time and expense inasmuch as the plan was to be sufficiently comprehensive and elaborate to take care of future conditions within a considerable radius of the bridge plaza. The City accordingly gave its approval to the bridge location without requiring the complete elaboration of such a plan and the working out of an agreement with the City covering the bridge approach.

The discussions and negotiations between representatives of the City and the Port Authority had gradually developed the viewpoint that future facilities to take care of the growth of bridge traffic should provide main outlets, other than those available by existing surface street connections, to Riverside Drive, Fort Washington Avenue, and Broadway. It was generally conceded that whenever the bridge traffic should increase to a volume of 10 000 000 vehicles annually, such outlets—particularly an outlet to points east of Broadway—would become imperative.

A study of traffic conditions was made by a special committee. Future bridge traffic and its distribution in the vicinity of the approach, the street capacities, and present and future local traffic were all analyzed. The Special Committee reported that under the original plan the approach street system would be taxed at once to or beyond its capacity by the addition of the bridge traffic, and that within approximately four years after the opening of the bridge extensive deficiencies in capacity would prevail on practically all streets within the affected area, with the exception of Riverside Drive. The Committee recommended, among other things, increased roadway facilities between the plaza and Riverside Drive, and a new facility between the plaza and Amsterdam Avenue. In line with the findings of the Special Committee, the Engineering Sub-Committee reported similar recommendations on November 11, 1927.

In the spring of 1928 the Port Authority and the City reached agreement as to the general construction of approaches to Riverside Drive and an extension of the approach to Amsterdam Avenue by means of a tunnel in West 178th Street. A committee composed of Arthur S. Tuttle, M. Am. Soc. C. E., Consulting Engineer of the Board of Estimate and Apportionment and O. H. Ammann, M. Am. Soc. C. E., Chief Engineer of the Port Authority, was appointed to develop detailed plans for the bridge approaches and highway connections.

The actual planning was done by the Engineering Staff of the Port Authority under the direction of the aforementioned Committee. A set of criteria governing the work was drawn up to meet the requirements of modern traffic, and was adhered to as far as was reasonably practicable. These criteria were as follows:

1.—An effort should be made to decentralize traffic lanes on the approaches, thus avoiding one central locus of congestion at a "bridgehead."

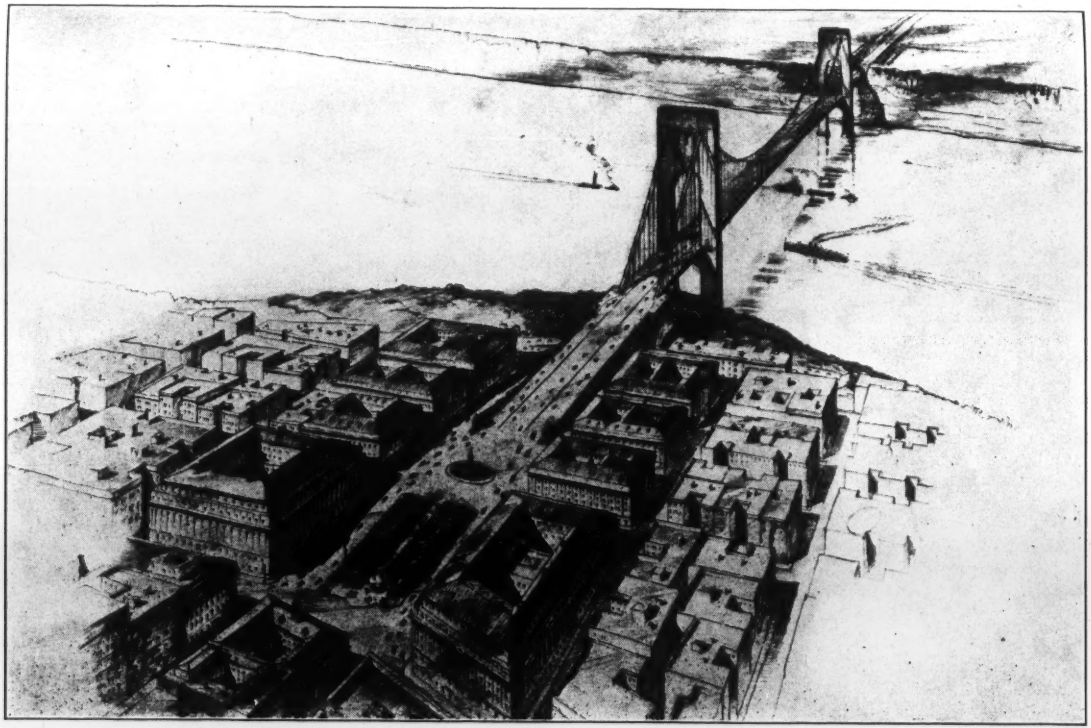


FIG. 1.—EARLY SCHEME FOR THE NEW YORK APPROACH, GEORGE WASHINGTON BRIDGE (INITIAL STAGE).

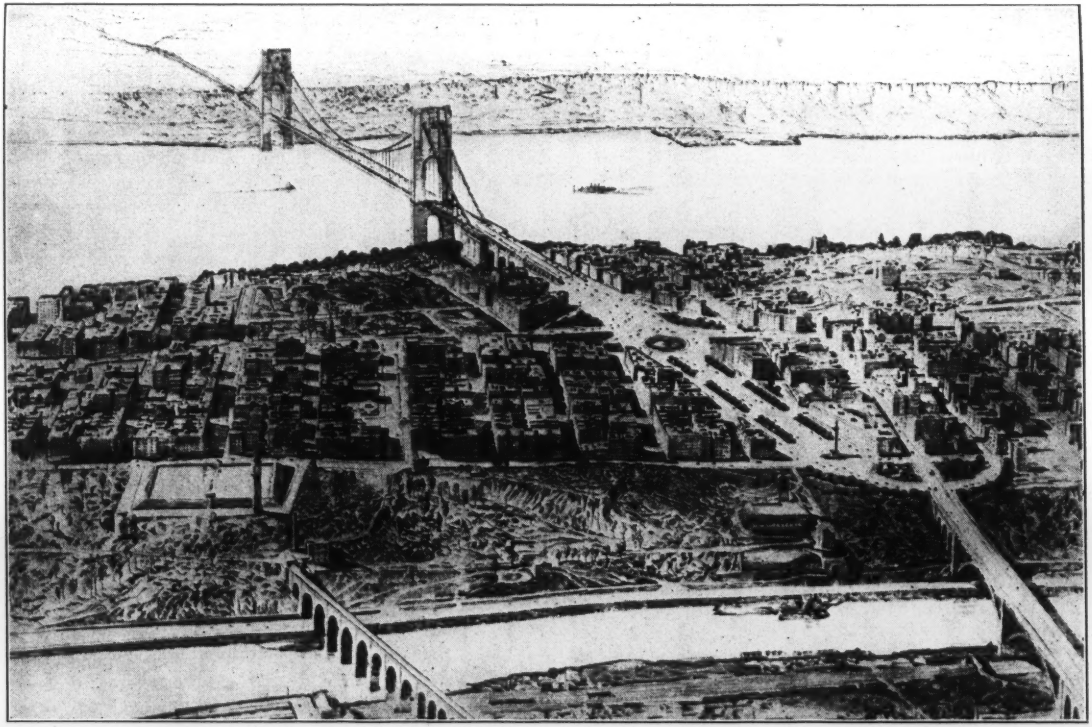


FIG. 2.—EARLY SCHEME FOR NEW YORK APPROACH, GEORGE WASHINGTON BRIDGE (ULTIMATE STAGE).

2.—There should be automatic traffic routing and control, rather than a complicated mechanical or electrical installation or an expensive personnel to direct bridge traffic.

3.—No crossings at grade of intercepting traffic, whether of street or bridge traffic, should be permitted.

4.—Left turns should be avoided.

5.—Provisions for toll collection should be eliminated on the New York approach, all tolls to be collected in New Jersey.

6.—All grades should be as flat as possible consistent with drainage and, in any event, should not exceed 4% on main approach and connections carrying truck traffic, and 6% on connections with Riverside Drive carrying passenger automobiles only.

7.—As far as possible any approach roadway should accommodate "one-way" traffic only.

8.—An approach roadway handling bridge traffic exclusively should be widened and banked on curves to allow for normal traffic speed.

9.—Bridge traffic should be separated from normal street traffic until it reached a point of connection with a through thoroughfare.

10.—In revising existing street lines or grades, an effort should be made to confine the damage to abutting property to a minimum.

11.—Construction should be in two stages, that for the initial stage to be such as to make possible its incorporation in the later stages without interruption of traffic during the later construction.

12.—The street map of Manhattan should be revised or changed no more than necessary.

13.—Bridge traffic should not be added to streets carrying an appreciable amount of ordinary street traffic.

14.—All new construction of roadways should be built for the ultimate capacity of the bridge.

The Committee presented two reports; the first, on April 4, 1929, proposed a plan essentially like that ultimately followed, but mentioned the possibility of constructing a depressed roadway instead of a tunnel for the connection with Amsterdam Avenue. The Committee recommended that the cross-town roadway be built when traffic should reach 10 000 000 vehicles annually and funds should become available. The latter stipulation was made because of the limited amount of funds available from the original financing, the original New York approach plans having contemplated nothing beyond the plaza at Fort Washington Avenue.

The City recognized the position of the Port Authority, but was so impressed with the necessity of having the tunnel approach ready at the time of opening the bridge that it decided to arrange to finance and build the tunnel, with the agreement that the Port Authority would accept it when traffic should reach 10 000 000 vehicles annually, but not later than three years after the completion of the tunnel, when the Port Authority would pay the cost of the tunnel, not to exceed $2 500 000.

The Committee of Engineers continued its work and prepared detailed plans and estimates for a four-lane tunnel. These plans developed the fact that the tunnel would require an expensive ventilation system and would cost approximately $6 000 000, much more than had been anticipated. It was out of the question for the City to proceed with the work at that figure.

Studies were then made of an open depressed roadway, which had been mentioned as an alternative in the report of April 4, 1929, of the Engineering Committee. The great disadvantage of the depressed roadway was the amount of property required. The City objected to the destruction of so much property value.

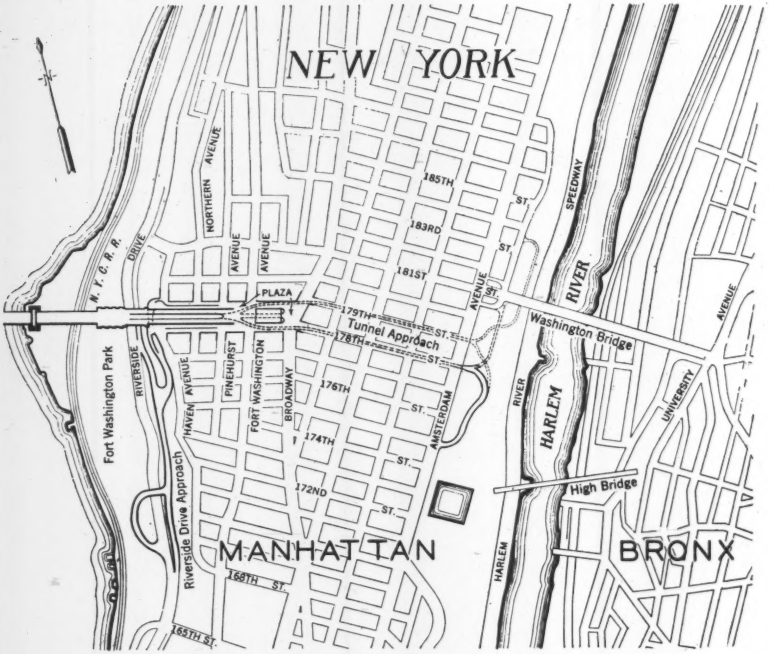


FIG. 3.—GENERAL PLAN OF NEW YORK APPROACH AND CONNECTIONS.

After much study of the situation, the Committee rendered a report on March 17, 1930, recommending a method of procedure which would provide the necessary facilities. In brief, it was proposed that for the initial traffic condition the Port Authority construct a two-lane, two-way tunnel in West 178th Street and widen West 178th and West 179th Streets, between Fort Washington Avenue and Broadway, to 36 ft by setting back the curbs. For the final traffic condition a second two-way tunnel was to be constructed in West 179th Street. The Port Authority agreed to the plan (Fig. 3) and further agreed to complete the approach roadways between Fort Washington

Avenue and Haven Avenue and the Riverside Drive connections west of Haven Avenue for the initial traffic condition.

In order to make it financially feasible for the Port Authority to build the tunnel under 178th Street, certain modifications in the procedure for carrying out the plan for the facilities west of Broadway were required. A plan was devised whereby approximately $4 500 000 worth of work could be deferred, to be done later as funds should become available, and well in advance of the traffic requirements. This procedure would relieve the City wholly of the necessity of financing any part of the work.

The plan provided that after the bridge should have been opened six months to traffic, the Port Authority would begin the construction of the two-lane vehicular tunnel under West 179th Street, completing it within eighteen months thereafter.

Within two years after the bridge traffic would have reached 10 000 000 vehicles in a preceding twelve months, the Port Authority would make a number of other improvements, such as:

(*a*) Complete the work required to widen West 178th Street and West 179th Street, between Fort Washington Avenue and Broadway, to 80 ft.

(*b*) Build a plaza on the west side of Broadway, between West 178th Street and West 179th Street.

(*c*) Build the ramp from the lower roadway of the bridge approach at Fort Washington Avenue to the surface plaza on the west side of Broadway.

(*d*) Complete the bridge approach ramp, from the anchorage to Fort Washington Avenue, to its full width.

(*e*) Complete the surface plaza between Pinehurst Avenue and Fort Washington Avenue.

Because of the impossibility of foreseeing the extent of the future traffic requirements of the bridge, in addition to that of the four initial bridge lanes, the Port Authority has agreed with the City to refrain from adding to the initial lanes until it shall be mutually determined through joint study whether additional street connections are necessary, and pending an agreement between the City and the Port Authority as to terms and conditions.

Description of the New York Approach

Reference has been made to the fact that the New York approach plans were modified to permit the initial construction of a certain part of the work prior to the opening of the George Washington Bridge to traffic and the remainder of the work necessary to complete the bridge approaches, according to the plan approved by the Board of Estimate and Apportionment of the City of New York, the Port Authority, and the Governor of the State of New York, at a later date.

The plan may be better understood by reference to Figs. 4 and 5. It embraces the following principal elements:

1.—The main approach ramps from the bridge proper to the plaza area west of Fort Washington Avenue and the side ramps and street connections beside the main ramp as far west as Haven Avenue.

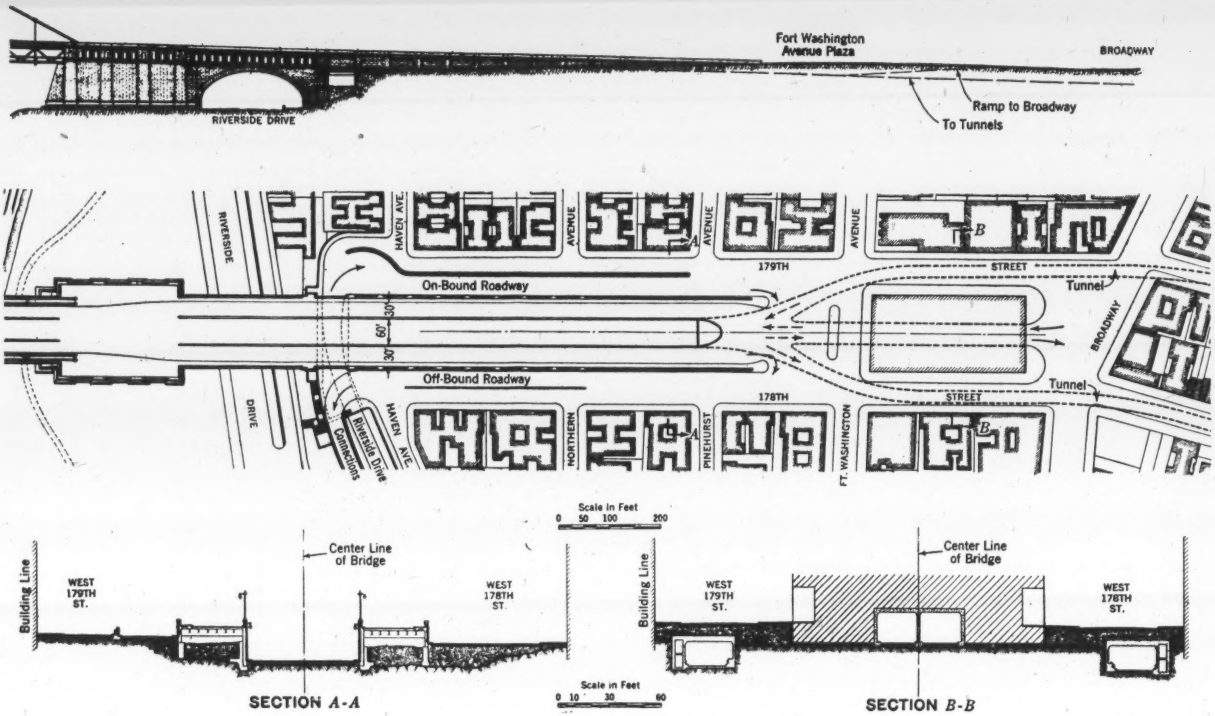


FIG. 4.—NEW YORK APPROACH, GEORGE WASHINGTON BRIDGE, ANCHORAGE TO BROADWAY, FINAL CONDITION.

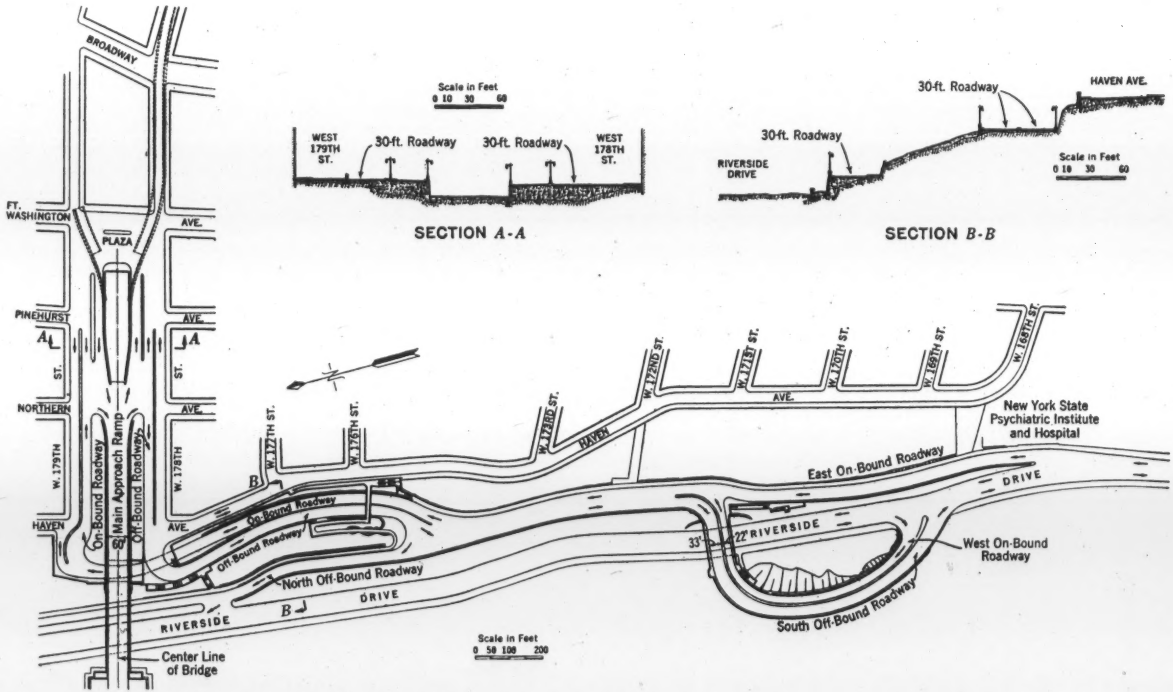


FIG. 5.—NEW YORK APPROACH, INITIAL CONDITION, INCLUDING RIVERSIDE DRIVE CONNECTIONS.

2.—The approach connections with Riverside Drive from their points of junction with the side ramps or roadways at Haven Avenue.

3.—The improvements in the so-called "Broadway Block" bounded by Broadway, Fort Washington Avenue, West 178th Street, and West 179th Street.

4.—The tunnel approaches to Highbridge Park, east of Amsterdam Avenue.

The terms, "initial" and "final" construction of the New York approach, refer only to the provisions for handling the traffic of the upper deck of the main structure. The lower deck, which may be added to accommodate four lines of rapid transit traffic if required, at a later date, will make connection with a ramp passing through the anchorage and over the arch across Riverside Drive at a level immediately below the vehicular traffic ramps, and so into its connection with whatever subway system may be planned at the time. It should be noted that construction has been planned so as to permit suitable future development of such an approach without alteration to the supports of the surface ramps. The plan will be considered under the aforementioned four headings.

1.—Main Approach Ramps to Fort Washington Avenue.—According to the plan for the "final condition," as shown on Fig. 4, the approach ramp is separated into three roadways, the central of which is 60 ft in width, while the roadways on either side are 30 ft wide. Sidewalks, 6 ft wide, are located beside the side roadways. All three roadways start at a common level at the anchorage in Fort Washington Park. The central roadway descends eastward on a 4% grade, attaining the level of the ground surface at Northern Avenue, and continues thence eastward on a 1.82% grade to a sub-surface plaza at Fort Washington Avenue. The two 30-ft outside roadways slope down more gradually, on a 1.2% grade, to a surface plaza between Pinehurst Avenue and Fort Washington Avenue, directly above the sub-surface plaza.

Vehicles to and from Broadway and the cross-town tunnels will use the central roadway and the sub-surface plaza. Other traffic may make direct connection with the adjacent surface streets, vehicles to and from Riverside Drive utilizing the 30-ft roadways which follow the general ground surface and parallel the main ramp as far as Haven Avenue.

At Northern Avenue, where the central roadway attains ground level, direct connections from it to the Riverside Drive approach roadways are provided, passing under the side roadways of the main ramp.

For initial conditions, as shown on Figs. 5 and 6, the central roadway only has been constructed. The outside roadways of the main ramp are omitted entirely east of the anchorage, the space which they are designed to occupy between Service Street and Northern Avenue being for the present merely graded areas. The central roadway receives and discharges surface traffic at Northern Avenue, all traffic except that to, or from, Riverside Drive, passing over the plaza at Fort Washington Avenue. The Riverside

FIG. 6.—VIEW OF TRAFFIC OPERATION ON NEW YORK APPROACH, LOOKING EAST FROM TOWER.

FIG. 7.—VIEW OF APPROACH CONNECTIONS FROM RIVERSIDE DRIVE, GEORGE WASHINGTON BRIDGE.

Drive traffic "offbound" turns at Northern Avenue into the 30-ft connecting roadway running north of, and parallel to, 178th Street as far as Haven Avenue. The "onbound" Riverside Drive traffic traverses a similar roadway parallel to 179th Street. East of Northern Avenue the central roadway continues to an entrance to the sub-surface plaza and the tunnel under West 178th Street.

2.—Approach Connections with Riverside Drive.—Riverside Drive is more than 115 ft lower than the ramp roadways at the point of passage of the latter over the Drive. To make connection with the Drive the approach roadways west of Haven Avenue follow the hillside southward as shown on Figs. 5, 7, and 8. These connections are designed to effect complete separation of the "onbound" and "offbound" traffic and to receive and discharge traffic at Riverside Drive without grade crossings.

The roadway that accommodates "onbound' traffic coming from points south by way of Riverside Drive begins at a point approximately 200 ft south of 168th Street and extends northward following the line and gradient of the former Service Street, crosses under the bridge approach structure, and joins the approach roadway paralleling West 179th Street at Haven Avenue.

The roadway that accommodates bridge traffic coming from points north on Riverside Drive begins at a point on the west side of the Drive approximately opposite 169th Street. Traffic enters the roadway by a right turn from the Drive, follows the roadway as it curves north, passes over Riverside Drive by way of an arch opposite West 171st Street, and there joins the other branch of the "onbound" roadway previously described.

The roadway connection carrying "offbound" bridge traffic to Riverside Drive begins at Haven Avenue, at the end of the approach roadway that runs parallel to West 178th Street and extends in a southern direction, parallel with the "onbound" roadway, to a point approximately opposite West 176th Street where the roadway forks. The traffic intending to travel southbound on Riverside Drive continues on a branch that crosses over the Drive on the lower arch, previously described, and thence southward through Fort Washington Park to a junction with Riverside Drive at a point approximately opposite West 168th Street. Traffic northbound takes the roadway which turns north at the fork and enters Riverside Drive at the foot of West 177th Street.

The "onbound" and "offbound" roadways comprising the connections with Riverside Drive are 30 ft wide to accommodate three lanes of traffic, except in the case of the "onbound" roadway south of 172d Street, which has a width of 35 ft to provide for parking facilities, and the "onbound" roadway for southbound bridge traffic on Riverside Drive which is 20 ft wide.

3.—Improvements Between Fort Washington Avenue and Broadway.—From the sub-surface plaza under Fort Washington Avenue, the final plan contemplates a ramp 40 ft wide, rising to a proposed plaza on the west side of Broadway (Fig. 4). From the same sub-surface plaza, "offbound" traffic may enter the vehicular tunnel under 178th Street. "Onbound" traffic will come to the plaza from a similar tunnel under 179th Street.

In order to provide ample street capacity in this area, West 178th Street and West 179th Street are to be 80 ft wide. The plan also proposes in this block, a building suitable for the location. The structure would have arcaded sidewalks inside the building lines, making possible curb locations to provide roadways 63 ft wide.

Under initial conditions, only the tunnel in West 178th Street connects with the plaza under Fort Washington Avenue. West 178th Street is widened to 36 ft between curbs, but the buildings in the block between this street and West 179th Street west of Broadway are undisturbed.

4.—Tunnel Connections to Highbridge Park.—The vehicular tunnels under West 178th Street and West 179th Street, connecting with the subsurface plaza at Fort Washington Avenue, have been mentioned previously. These tunnels, each two lanes in width, are designed to carry traffic to Amsterdam Avenue, the Harlem River Speedway, the Washington Bridge over the Harlem, and to any other crossing over the Harlem River that may be designed later. For the initial traffic condition the tunnel under West 178th Street only is being completed. A roadway from the portal in Highbridge Park just east of Amsterdam Avenue is being constructed by the Port Authority to connect with Amsterdam Avenue. The tunnel is to carry a single lane of traffic in each direction pending the completion of the second tunnel.

Approach Structures

One of the most interesting features of the New York approach is the arch that supports the main ramp over Riverside Drive. It has a clear span of 196 ft, a rise of 36 ft, and a clear height above the Drive of 66 ft at the crown. It is of the barrel type and is constructed of reinforced concrete. Fig. 9 is a view taken April 10, 1931, during the progress of construction. The arch is designed for a load of 235 000 lb per lin ft, such a high loading resulting largely from the provision made for a future lower deck capable of carrying four tracks of rapid transit.

The arch thrust of the west abutment is resisted by the anchorage of the main bridge. At the east abutment the thrust is transmitted into the rocky hillside through a comparatively small concrete block. The barrel design was chosen in preference to an arch with ribs because of the flexibility thus made possible in the location of railway tracks and because the barrel design made simplified construction possible. Under final conditions, the structure will be 145 ft wide, but initially the barrel width is limited to 65 ft. It varies in thickness from 7.5 ft at the springing line to 4 ft at the crown.

The superstructure above the arch barrel is a combination of concrete and steel. Concrete cross-walls spaced 24 ft apart were poured integrally with the arch barrel. These walls were brought to a level about 3 ft below the grade of the future lower-level ramp. They carry a steel superstructure of columns supporting plate girder cross-beams and suitable framing supporting a 9-in. concrete slab roadway reinforced with 6-in. bar trusses. The structure above the arch barrel is masked for the present with spandrel walls of mortar stucco on wire mesh. With the addition of the side roadways

FIG. 8.—VIEW OF APPROACH CONNECTIONS TO RIVERSIDE DRIVE, LOOKING SOUTH FROM MAIN ROADWAY.

FIG. 9.—VIEW OF ARCH FOR MAIN RAMP OVER RIVERSIDE DRIVE, DURING CONSTRUCTION, APRIL 10, 1931.

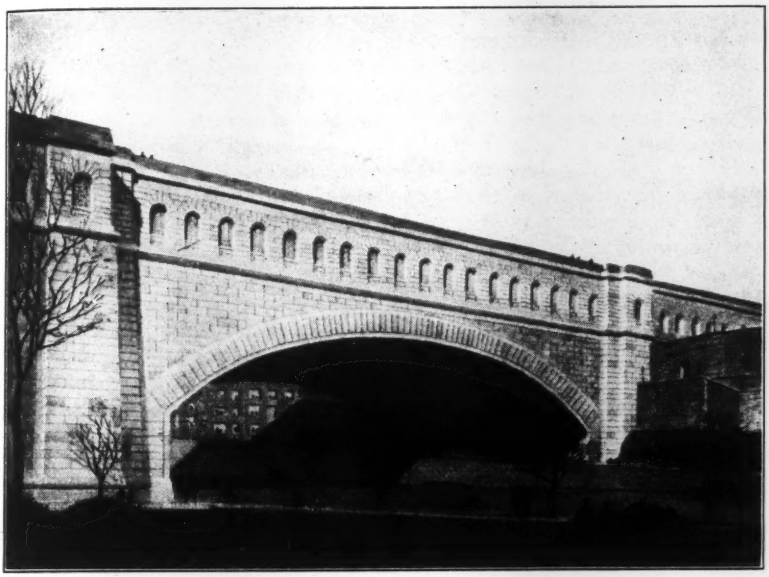


FIG. 10.—PERSPECTIVE VIEW OF MAIN ARCH OVER RIVERSIDE DRIVE.

FIG. 11.—LOWER ARCH OVER RIVERSIDE DRIVE.

and the widening of the arch to 145 ft, the superstructure will be faced with granite, as will the abutments and the remainder of the main approach ramp. A perspective view of the completed arch is shown on Fig. 10.

The main ramp contains no unusual type of construction, other than the arch, and consists of conventional braced steel framing on a concrete column substructure. The "onbound" roadway from Riverside Drive passes under the main ramp west of Haven Avenue and, at this point, the ramp roadway is carried by 92-in. plate girders having spans of approximately 66 ft.

The major part of the roadway system connecting with Riverside Drive is of the retaining-wall type, built into the hillside. Near the north end, however, a section of the "offbound" roadway (shown in Fig. 7), is designed as a rigid frame structure to resist the earth pressure. This design obviated the necessity of a gravity retaining wall which would have projected into the area desired for roadway use.

The lower arch over Riverside Drive (Fig. 11) is of reinforced concrete faced with stone; it has a span of 120 ft. The arch barrel is 75 ft wide. It is comparatively flat and utilizes a broken joint at the center of the span which relieves the arch from any floor action or wall action. The thrust of the arch on the east side is transmitted to outcropping rock by means of a small abutment. At the west end, it was necessary to construct a concrete abutment, 40 ft deep, 26 ft wide, and 80 ft long, because of the depth of rock satisfactory for foundation purposes.

The approach structure below the arch and on the west side of Riverside Drive is of fill within retaining walls. The retaining walls themselves were placed on fill and required continuous pouring of the footings in order to insure uniform settlement. Joints are provided at 50-ft intervals along the walls.

The tunnel in West 178th Street is built on the usual lines of subway construction. A cross-section, shown on Fig. 12, indicates that, in section, the tunnel is a concrete box about 40 ft wide and 20 ft high in outside dimensions. The steel framing is made up of bents, 5 ft center to center, which are encased in the concrete. The roadway is 22 ft in width with a clear height of 14 ft. Ducts for fresh air and for vitiated air are on one side, the fresh air duct being below the vitiated air duct. Transverse ventilation is provided, fresh air being admitted to the tunnel immediately above the curb on both sides of the roadway and the vitiated air removed through ports in the ceiling. A ventilation building for the tunnel in 178th Street, constructed for the initial traffic conditions, is situated about midway of the tunnel length.

The pavement on the main approach ramp is a reinforced concrete slab of 9-in. thickness. The approach roadways to Riverside Drive, where they parallel the main ramp, and down to the point of separation of the "onbound" roadways, are paved with granite block; south of that point they are of sheet asphalt on a concrete base, except on the curves where asphalt block paving has been used because of its non-skid properties. The tunnels in 178th and 179th Streets are to be paved with granite block.

Development of the New Jersey Approach Plan

The approach plan originally submitted to the Borough of Fort Lee and the Governor of New Jersey is shown on Fig. 13. It embraced essentially the approach proper from the face of the cliffs to Lemoine Avenue. The approach was to extend over Hudson Terrace and then was to be widened into a spacious toll-collection area, this area continuing to Lemoine Avenue to terminate in a traffic circle. The plan included highway connections with Hudson Terrace and local marginal streets along the approach, which, in turn, were to

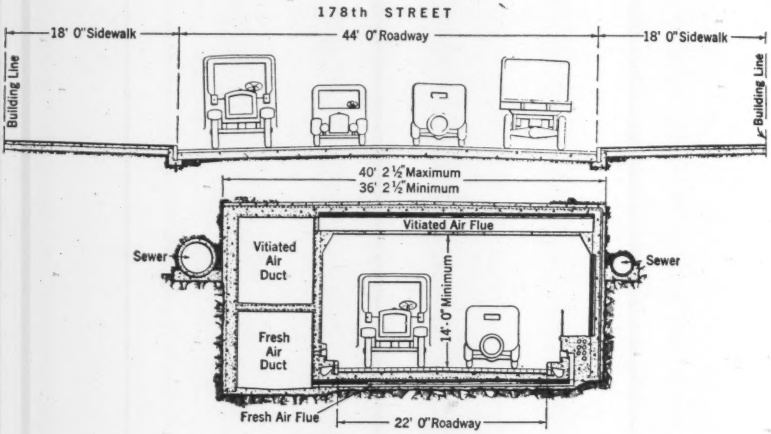


Fig. 12.—Cross-Section of Tunnel in West 178th Street, New York, N. Y.

connect with all intersecting local streets. In submitting the plan, the Port Authority offered to undertake all necessary changes and improvements within the area bounded by the marginal streets, imposing no expenditures on the Borough of Fort Lee.

The plan was approved by the Borough of Fort Lee and by Governor A. Harry Moore, of New Jersey, in the spring of 1927. At that time it was understood that Lemoine Avenue would be a State highway, improved and widened by the State, and would thus form, at least initially, the principal highway connecting with the bridge approach. It was also understood that, depending upon what the State Highway Commission might do in the way of improving Lemoine Avenue, or building new connecting highways, the plan approved by the Borough of Fort Lee and Governor Moore would have to be re-studied and probably modified in the vicinity of Lemoine Avenue.

An Engineering Committee composed of representatives of the State Highway Commission, Bergen County, the Borough of Fort Lee, and the Port Authority, undertook a thorough study of the connections. A large number of possibilites were studied, including those for grade separation at Lemoine Avenue, and for the extension of a wide boulevard west of the plaza

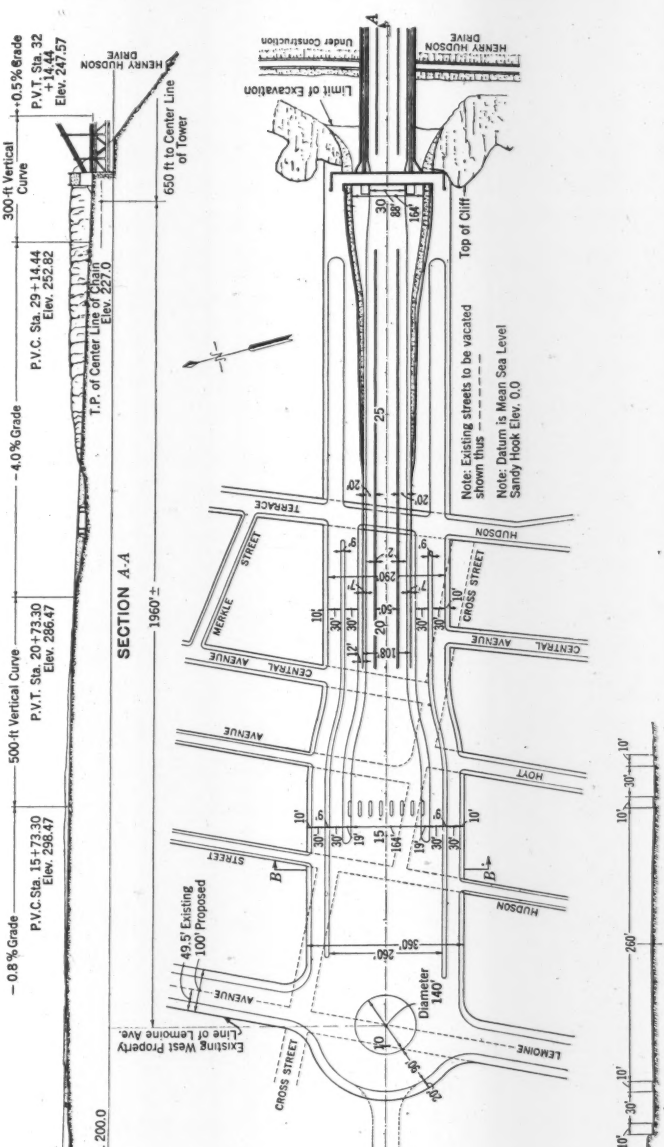


FIG. 13.—ORIGINAL PLAN OF NEW JERSEY APPROACH AS SUBMITTED.

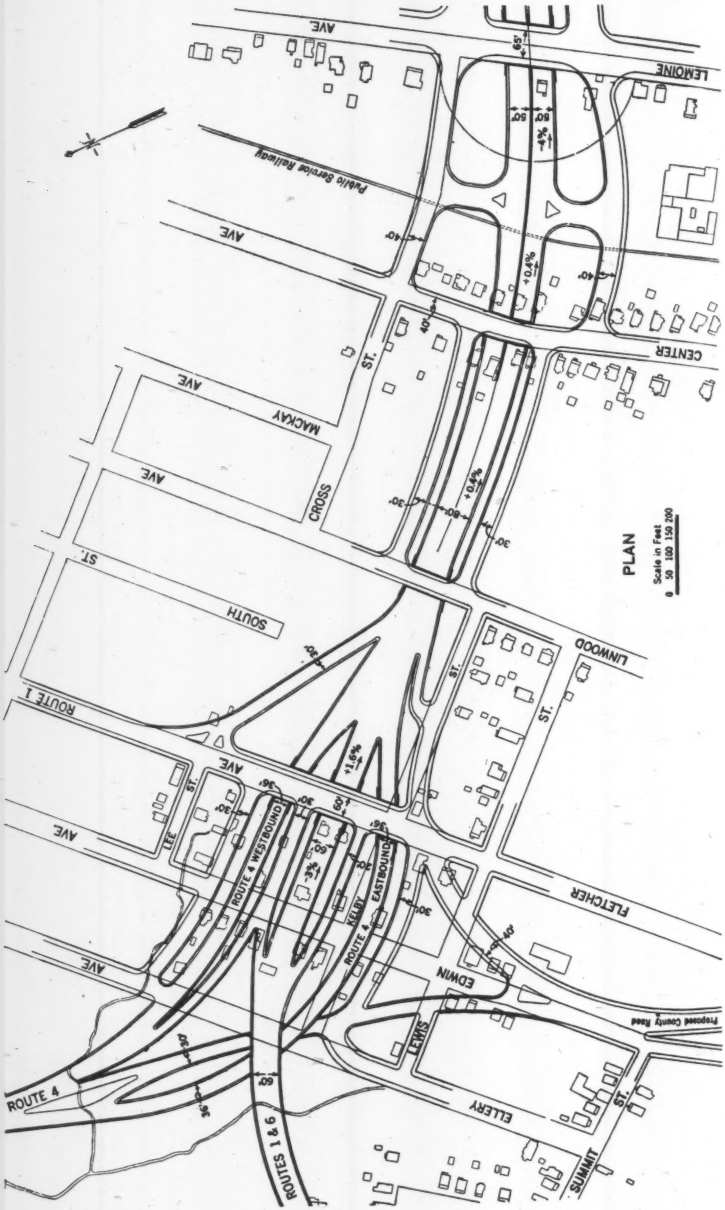
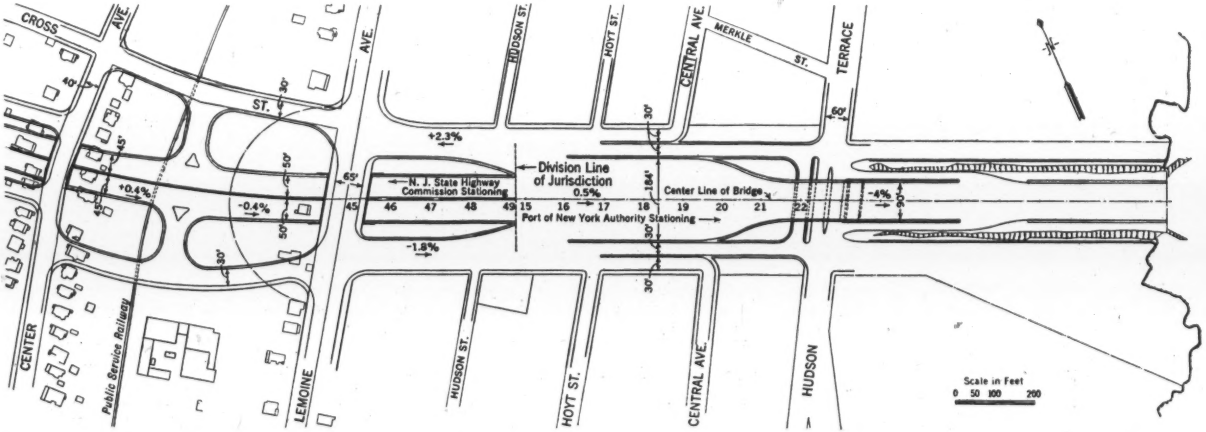
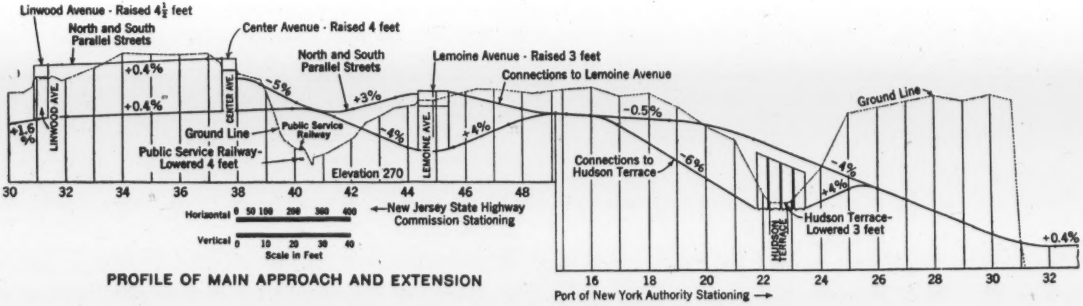


FIG. 14.—FINAL PLAN OF





PROFILE OF MAIN APPROACH AND EXTENSION

to connect with arteries serving the Hackensack, Paterson, and Passaic areas. During the course of these studies the State Highway Commission developed a system of new and improved highways west of the bridge plaza designed to serve local, through, and bridge traffic.

In the early development of the plans it was intended that the work of the Port Authority should terminate at Lemoine Avenue and that the State should undertake the work to the west. However, as the negotiations between the State Highway Commission and the Port Authority progressed, the point of view was evolved that the proposed highway from Lemoine Avenue to the connection with State Highway Route No. 4, near Fletcher Avenue, a length of about 2 000 ft, should be considered at least partly as an essential feature of the bridge approach system, necessary to maintain traffic flow to and from the structure, and properly should be built by the Port Authority.

The plan of the Port Authority as approved by both the Borough of Fort Lee and Governor Moore, and used as a basis for financing, had provided for the termination of the approach at Lemoine Avenue. Thus, no funds were available to meet the cost of the extension, estimated at $2 500 000.

The Port Authority then proposed that it would undertake to build the extension when the bridge traffic should reach 10 000 000 vehicles per year and as funds should become available, or that the State proceed to build the extension as part of the proposed State Highway, in which event the Port Authority would agree to reimburse the State to the extent of $2 500 000 when the bridge traffic should reach 10 000 000 vehicles per year and as the funds should become available. The latter alternative was finally adopted.

On December 27, 1929, the Engineering Committee submitted a report recommending a plan, shown on Fig. 14, "considered to be best suited to all conditions." The report, which is signed by J. L. Bauer, M. Am. Soc. C. E., State Highway Engineer of the New Jersey State Highway Commission (successor to William G. Sloan, M. Am. Soc. C. E.), R. P. McClave, Engineer of Bergen County, S. Wood McClave, M. Am. Soc. C. E., Engineer of the Borough of Fort Lee, and O. H. Ammann, M. Am. Soc. C. E., Chief Engineer of the Port Authority, states, in part:

"Safe and expedient handling of traffic has been the paramount aim in developing the plan, but due consideration has also been given the aesthetic effect of the plan on the landscape. An effort has also been made to retain existing conditions and the disturbance of alignment and grades of existing highways has been minimized. All gradients on major routes have been held to a 4% maximum. Certain minor connections have a steeper gradient, but all are well within easy operating possibilities.

"The plan provides for adequate facilities not only for the initial four-lane traffic over the bridge, but for the final eight-lane capacity of the upper deck and, for additional lanes on the lower deck, if necessary. The plan has been developed so that future modifications can be consummated with little or no disturbance to traffic."

To facilitate the actual work of construction an arrangement was made whereby the work done by the State Highway Commission would extend to a line about 470 ft east of the west street line of Lemoine Avenue. However, since the Port Authority had taken into consideration in its original plan

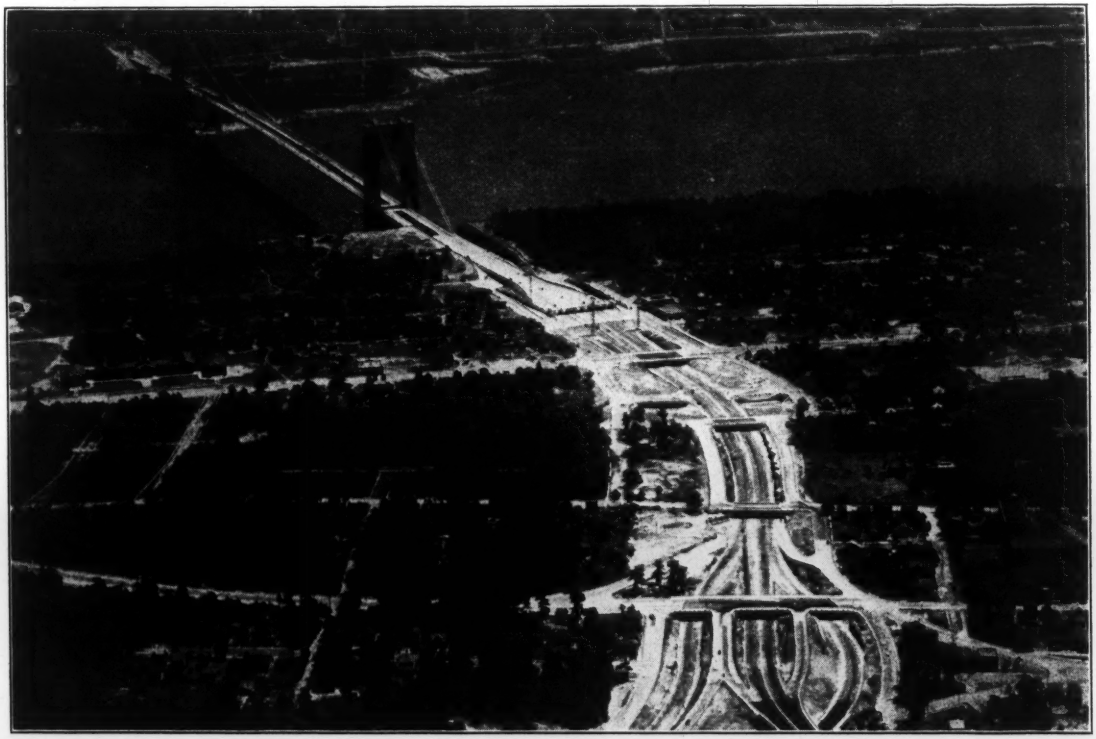


FIG. 15.—AERIAL VIEW OF NEW JERSEY APPROACH, GEORGE WASHINGTON BRIDGE.

and cost estimates the approach to, and a plaza at, Lemoine Avenue the arrangement therefore included payment by the Port Authority of the cost of the work from the division line to the west side of Lemoine Avenue.

The reference which has been made to the $2 500 000 cost of the extension to Fletcher Avenue should not be understood to include the entire expenditure on the new roadways built by the State Highway Commission in that area, but merely the part of the cost directly chargeable to bridge traffic only and to be borne by the Port Authority. The entire system aggregated an additional cost of several million dollars.

Description of the New Jersey Approach

The New Jersey approach (Fig. 15), where it joins the bridge proper is in an open cut approximately 50 ft below the top of the Palisades at the face of the cliff. Continuing westward, the main ramp, which is 90 ft wide, rises on a grade of 4%, attains the ground surface in a distance of approximately 600 ft, and thence passes over Hudson Terrace, beyond which the ramp is widened into a spacious toll-collection area. From the toll area direct street connections are made with Lemoine Avenue, Hudson Terrace, and intermediate streets. From the east side of Hudson Terrace special ramps to the main bridge approach make additional direct connection between that street and the bridge.

As a continuation of the main ramp, a concrete highway, 100 ft wide, passes under Lemoine Avenue and thence continues as a depressed roadway under Center Avenue and Linwood Avenue where it is narrowed to 90 ft. At Fletcher Avenue, the main ramp is divided into a number of direct connections with State routes which have been converged in that area.

These connections are designed to eliminate all grade crossings and to form a safe collecting and discharging agency for high-speed traffic. Cross-street connections within the Borough of Fort Lee are provided by means of ramps including a "clover leaf" layout between Lemoine Avenue and Center Avenue. Marginal streets between Center Avenue and Linwood Avenue connect with the "clover leaf" system.

The approach ramps, plaza, and connecting roadways have been built initially to the full width for the accommodation of ultimate bridge capacity, including any vehicular capacity which the lower deck may have if, and when, added at a future date.

In the design of the New Jersey approach access to the lower deck of the bridge has been considered only to such an extent as to insure that any construction incorporated in the original design shall in no way restrict connections to the lower deck at some future time.

New Jersey Approach Structures

The open cut through the Palisades was excavated as part of a contract which also included excavation of pits and tunnels for anchoring the cables. The cut in rock is 146 ft wide throughout its length and involved the excavation of approximately 200 000 cu yd of material. The only viaduct structure within the limits of construction by the Port Authority is the over-

head crossing at Hudson Terrace where the approach ramp (90 ft. wide) is supported by a structure 135 ft long, consisting of three transverse steel bents of five columns each.

The excavation for the ramps to the west side of Hudson Terrace and for the widening of the plaza between Hudson Terrace and Lemoine Avenue required the removal of more than 75 000 cu yd of rock. The entire roadway ramp and plaza area is paved with concrete, with a slab thickness of 9 in. The construction west of the line of jurisdiction, separating the Port Authority and State Highway work, which has been mentioned previously, was done by the State Highway Commission.

Provision for Toll Collection

The facilities for the collection of tolls are at the Fort Lee end of the bridge in a spacious area west of Hudson Terrace and at the two side ramps leading eastward from Hudson Terrace to the bridge. A field office and garage are centrally situated just south of the toll area. The two are combined into a single structure of fireproof construction with an exterior finish of granite and sandstone rubble masonry.

The buildings for the collection of vehicular tolls extend across the plaza opposite the field office (Fig. 16). Seven pairs of booths serve fourteen lanes. A toll-house at either end of the line of toll booths serves an additional lane each, making a total of sixteen toll lanes. All the toll booths are connected by a continuous canopy which extends from toll-house to toll-house over the entire line. Provision for vehicular tolls is also made at the ramps leading eastward from Hudson Terrace to the main bridge ramp.

The toll-houses are two-story structures, 40 ft long and 12 ft 6 in. wide, of fireproof construction of granite exterior. These houses provide locker-rooms and showers for the collectors and also furnish space for an instrument room, a battery room, and rooms for tellers, collectors, and police sergeants. Heat is furnished for the toll booths and toll-houses from the field office.

The toll booths are also of substantial fireproof construction with frames of structural steel. These booths are plastered inside and are finished on the outside with sheet aluminum for architectural effect. Toll booths serving the ramps leading eastward from Hudson Terrace are, in general design, similar to the main group. A single pair of booths accommodates two lanes in each ramp. The field office and garage, toll-houses, and toll booths have been treated architecturally to conform to the general dignity of the structure.

Plaza Illumination

A special condition was encountered at the New Jersey toll-collection area where a space approximately 450 ft long by 250 ft wide had to be illuminated adequately without offering obstruction of any sort to vehicular travel. To meet this condition, four flood-light towers are placed near the corners of the area. Two of these towers are approximately 185 ft east of the line of the toll booths and two are approximately 280 ft west of the toll buildings in the area.

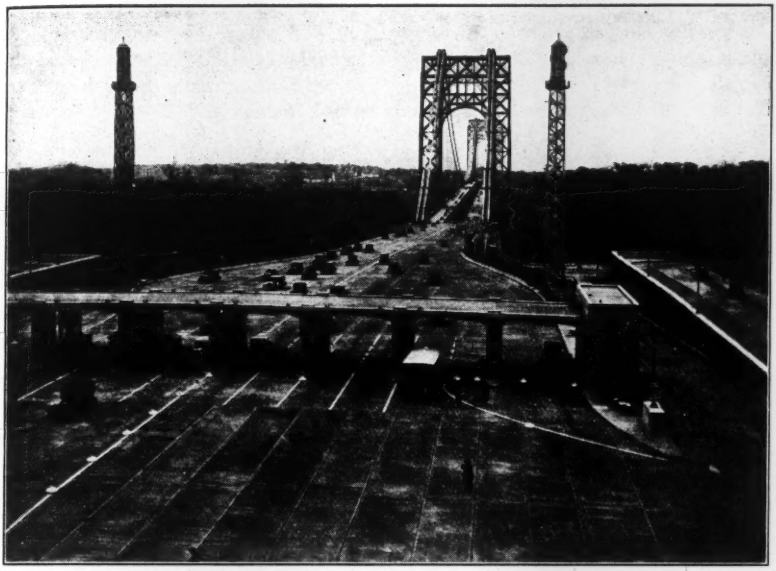


FIG. 16.—TOLL COLLECTION FACILITIES, NEW JERSEY APPROACH, GEORGE WASHINGTON BRIDGE.

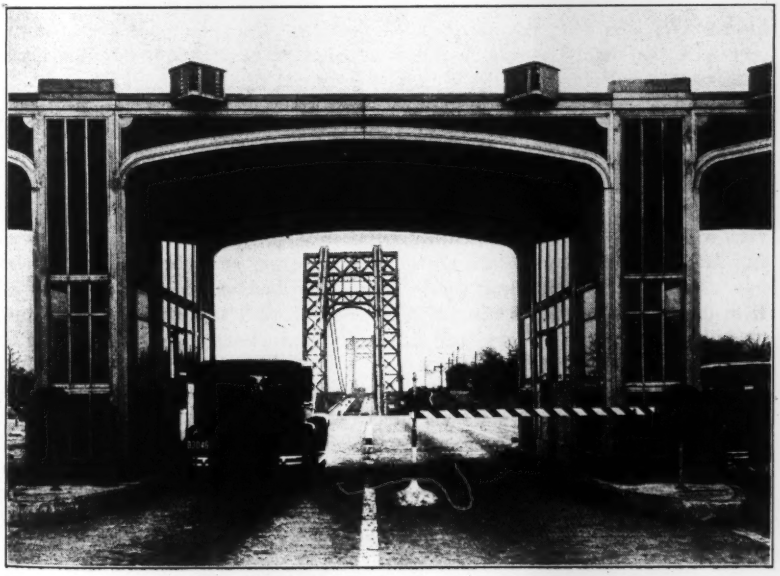


FIG. 17.—CLOSE-UP VIEW OF TOLL BOOTHS.

These towers are equipped with batteries of flood lights that illuminate the plaza brilliantly from a height of about 100 ft. The towers are of simple structural steel construction mounted on granite bases. They are painted to harmonize with the general bridge structure.

Vehicular Toll Collection Equipment

Automatic registering equipment has been installed at the toll booths. This equipment consists of four main elements:

(1) Fare registers for recording the amount of tolls.

(2) Tariff signs for indicating the classification of vehicles as registered by the toll collectors.

(3) Central office indicator boards for remote checking of vehicle classifications as registered by the collectors.

(4) Vehicular treadles for counting and recording the axles passing through each toll lane.

The tariff signs, indicator boards, and fare registers are inter-connected electrically while the vehicular treadles are electrically independent. These devices function as follows: When a vehicle enters a toll lane the operator stops momentarily at the booth to pay his toll. The collector receives the fare, makes change as necessary, and permanently records the transaction in printed form on the register according to classification of vehicle and amount paid. He accomplishes this by two simple motions of an actuating lever. The manipulation of the fare register also causes illuminated figures representing the classification of the vehicle to appear on a pair of overhead tariff signs erected at the entrance and exit of the toll lane (see Fig. 17). These signs are placed and arranged so that they are visible from practically all points on the plaza, such an arrangement giving the officers, or others in charge of the collection of tolls, a ready means of checking.

The officer in charge has a further check upon the collection of tolls, because when the collector registers a vehicle, the classification number (the same number as indicated on the overhead signs) is shown on one or more panel boards located centrally in near-by toll-houses, or in the field office near windows overlooking the plaza. These boards are designed to permit simultaneous checking of several collectors. They are distinguished from the tariff signs by the fact that, unlike the latter, they operate only at the will of the checker who throws a small switch for each lane that he wishes to observe.

As far as the operator of the vehicle is concerned, the toll collection operation is complete with the payment and the registration of the fare. No tickets are issued to be collected later. Instead, as the vehicle leaves the toll lane, it passes over a treadle in the pavement. The treadle operates electrically, counts each of the axles passing over it, and records these impulses by a mechanical printing operation in the fare register on the same sheet of paper with the classification count of vehicles. As vehicles are classified according to number of axles as well as to type and capacity, an exact check between the collector's record and the treadle count is possible.

All the aforementioned indicating and registering equipment has been designed specially for installation on this and other Port Authority toll projects and much of it represents pioneer work in this field.

Pedestrian Toll-Collection Facilities

Pedestrian tolls are collected on the approaches at either end of the bridge. Pedestrians pass through turnstiles housed in a suitable structure at the New Jersey anchorage. The building furnishes space for the cashier's office and for police purposes.

Similar facilities are placed at the east end of the arch over Riverside Drive where arrangements for pedestrian accommodation require passage through the interior of the arch structure to a stairway leading to the sidewalks at the anchorage.

Acknowledgments

It has been intended in this paper to indicate clearly the close co-operation with the various Municipal, County, and State agencies that was sought and received by the Port Authority.

In the development of the New York approach plan and in facilitation of construction operations the co-operation of Arthur S. Tuttle, M. Am. Soc. C. E., Consulting Engineer of the Board of Estimate and Apportionment of the City of New York, Clifford M. Pinckney, M. Am. Soc. C. E., Chief Engineer of the Borough of Manhattan, Robert Ridgway, Past-President, Am. Soc. C. E., Chief Engineer of the Board of Transportation, Edward A. Byrne, M. Am. Soc. C. E., Chief Engineer of the Department of Plant and Structures, and the late John R. Slattery, M. Am. Soc. C. E., Deputy Chief Engineer of the Board of Transportation, was very helpful. The following members of the Department of Water Supply, Gas, and Electricity—the Hon. John J. Dietz, Commissioner, Nicholas J. Kelly, Chief Engineer of Light and Power, William W. Brush, M. Am. Soc. C. E., Chief Engineer, Bureau of Water Supply—and numerous other officials of New York City Departments, have also rendered their hearty co-operation.

William G. Sloan, M. Am. Soc. C. E., former State Highway Engineer, and, later, J. L. Bauer, M. Am. Soc. C. E., the present State Highway Engineer of the New Jersey State Highway Commission, Hugh A. Kelly, Assoc. M. Am. Soc. C. E., R. P. McClave, County Engineer of Bergen County, and S. Wood McClave, M. Am. Soc. C. E., Engineer, Borough of Fort Lee, and other officials have co-operated with the Port Authority in the development and construction of the New Jersey approach.

Officials and members of the Public Utility Companies in both the State of New York and in the State of New Jersey, have rendered valuable assistance, as have also various civic bodies.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

TRANSACTIONS

Paper No. 1826

GEORGE WASHINGTON BRIDGE

DISCUSSION ON THE EIGHT PAPERS (NOS. 1818 TO 1825) DESCRIPTIVE OF THIS WORK

GENERAL CONCEPTION AND DEVELOPMENT OF DESIGN

DISCUSSION BY MESSRS. E. E. HOWARD, C. T. SCHWARZE, GUSTAV LINDENTHAL, HAROLD M. LEWIS, AND O. H. AMMANN

E. E. HOWARD,[1] M. AM. SOC. C. E. (by letter). In his paper Mr. Ammann has produced what may be termed an epic of engineering. The simple, lucid recital outlining the controlling conditions—governmental, financial, and physical—reviewing the many complex problems and analyzing them, and including in brief compass a comprehensive story of the entire undertaking makes the paper a notable one and worthy of the great structure it describes.

The feeling, even of the non-technical observer, that the completed bridge as it looms across the great river is "right and suitable," is a tribute to the constant reversion to the fundamentals of design so evident in the recital. That a structure so monumental should be of a hybrid type is unthinkable, yet every engineer who has been involved in the complexities of problems of unprecedented structures realizes how easily one may be deflected into specious solutions apparently offering marked advantages, which in the end result in disappointment.

The simplicity of the design is most striking in comparison with the various earlier proposals. It illustrates the modern trend toward structures with the fewest number of members adequately proportioned and properly placed. Studies of possibilities always lead off into complexities, but carried to conclusion they finally reach simplicity as the ultimate and proper solution. An outstanding case is the Golden Gate Bridge, at San Francisco, Calif., which was promoted for years as a combination cantilever and suspension arrangement, but in the final analysis has become a simple suspension span, doubtless influenced by the George Washington Bridge.

[1] Cons. Engr. (Ash-Howard-Needles & Tammen), Kansas City, Mo.

The history of various attempts to provide a bridge across the Hudson River is an interesting record. To earlier proponents there is the realization, not that their conceptions were at fault, but that the time was not yet ripe or the circumstances propitious. One must feel a sympathy for these pioneers of ideas.

Only with much more complete data, such as those published in the papers of this series, could any critical discussion of the soundness and wisdom of many determining conclusions be warranted. Nevertheless, in this paper there is evidence of such thorough and impartial consideration before reaching decisions, that any group of engineers, with all the facts before them, would substantially concur—except as such conclusions might be affected by personal preferences, or conceptions of beauty, or similar features not dependent on technical analysis.

Structures of great size are to the engineer, as the author indicates, only a question of materials and conditions and do not alter fundamentals. It is only important that size be commensurate with conditions, and it is as poor engineering to project a structure too large as too small. When mere maximum size, length, or height becomes a factor, good engineering shrinks into partial eclipse. The same care, thought, and management would be as notable if applied to a smaller structure.

The great center of population that is New York City and vicinity offers conditions widely different from any other section of the United States, or possibly of the world. The conclusion that the number of vehicles crossing the Hudson River should increase from about 5 000 000 in 1915 to 75 000 000 in 33 years seems rather incredible, but it is equally incredible that the number by actual count reached 25 500 000 by 1930—a period of 15 years. Considering the impression the vehicular traffic on the streets of New York City makes on the visitor one is inclined to wonder where 50 000 000 more vehicles on Manhattan Island are to operate.

The remarkable increases in vehicular traffic over the entire country from 1915 to 1930 were coincident with the period of multiplied production and wide diffusion of motor vehicles and the peak period of highway development. The realization of late years of the draft on the total National income of motor transportation again directs attention to the once much discussed "saturation point" in percentage of motor vehicles to population. Highway building will continue indefinitely, but doubtless at a more measured rate than for the period since the World War, which was somewhat comparable to the feverish era of railroad building which ended about a quarter of a century ago.

It would appear that other factors must enter in order to maintain the old rate of increase of vehicular movement. One such factor is the well-known and marked increase of local traffic with every mile of improved contiguous highway; but, evidently, the anticipated accelerating number of vehicle crossings of the Hudson River must arise in large measure from increasing population in the vicinity, and perhaps by encouragement of greater daily car use by additional and cheapened parking garages in New York City.

Increased crossings from long-distance travel probably cannot be anticipated as much of an element in the total traffic. Through highways for long-distance traveling are fairly well distributed over the country. Other and better roads will criss-cross counties and States and will develop and intensify local travel; but they will have relatively less effect on long-distance traffic—say, of 500-mile trips, or more.

The elaborate arrangement of approaches to the bridge is an answer as regards the immediate vicinity and an execellent illustration of proper consideration of methods of getting traffic to and away from a bridge. Public officials or, more particularly, prominent citizens promoting a bridge or viaduct which frequently will increase traffic near their property, sometimes resent cold facts of approach necessities—to be provided by appropriation of that same precious property or in other expensive ways. The writer could cite a case in which an engineer was eliminated because he recommended strongly against building a monumental structure, with a roadway 60 ft. wide, between two street ends built up with business buildings where parking, or at least stopping cars at curbs, was inevitable and where the width between curbs was only 40 ft., with no opportunity for side approaches. The suggestion that a structure only 40 ft. wide should be built and the money thus saved be used within a few years to build a parallel structure at another through street, a few blocks away, was received with disdain, evidently because it might divert traffic and build up business on the second street to the detriment of the first.

In New York City the problems are so great that the pressure of individual preference must be proportionately less than elsewhere. Experiences in smaller communities impel the melancholy conclusion that the attitude of the average good citizen toward the good of the entire community is largely measured by the Biblical axiom, "Where thy treasure is there will thy heart be also."

The excellent work of many State Highway Departments in which the Engineering Departments have relatively a free hand is accomplishing much and may be educating the public in the necessity of adequate approaches to main and restricted traffic arteries.

Of outstanding interest in design is the treatment of live load stiffening, and the author's discussion refers to early unstiffened bridges. Until a few years past there were in the Ozark Mountain country a number of so-called "home-made telephone-wire swing bridges" of spans up to 600 ft., or more, high above river gorges, which were of interest in that their crudity afforded such obvious economical efficiency. The cables were 6 in., or more, in diameter, of about 12-gauge galvanized wires simply laid together by hand, tied with occasional wire wrappings; and the hangers were frequently of loose strands of the same wires wrapped once around the cables and looped around the ends of floor-beams. The floors were a single layer of planks on wooden joists of one panel length, and provided for a single line of traffic. Although the bridges were wholly devoid of anything to function as stiffening trusses, deflections under the load of a heavy car or truck were surprisingly small, giving just enough feeling of sag to discourage speeding.

They were homely illustrations of the author's view that some arbitrary suspension bridge stiffening requirements may easily exceed practical necessities.

Another notable general feature of the Hudson River undertaking is the celerity with which it was carried forward to completion after a method of financing was finally established. The writer can recall no structure of comparative size with such variety of problems that has been brought to serviceable use so quickly. The tendency of public works enterprises to drag through controversies lagging through the Courts sometimes causes serious losses and tends to nullify the advantages of the improvement. As justice deferred may be justice denied, so public improvements begun, but unnecessarily delayed in completion, may result in detriment to a community. The writer could cite a large viaduct in a certain city which with controversies with railroads, hearings before Public Commissions, right-of-way condemnation suits, etc., is still in the Courts ten years after the community had determined the structure should be built. The detrimental effect upon adjacent property and upon other improvements can scarcely be estimated.

C. T. SCHWARZE,[2] M. AM. SOC. C. E. (by letter).—In this paper Mr. Ammann has made a distinct contribution in presenting a history of the many attempts to design a safe and adequate overhead crossing of the Hudson River, at New York City. The engineering difficulties involved in such an epoch-making design—both as to length of span and capacity for traffic—were, indeed, enormous. The splendid fruition of the adopted design is its own evidence of the care with which it was prepared, even to a consideration of background involving æsthetic feeling and, also, transportation development.

Aside from the actual kinds of structures considered in this paper, it is interesting to recall designs in non-structural steel arches that have been advanced at both ends of the chronologic period of North River bridge planning. Probably one of the first of such designs (in the earlier part of the Nineteenth Century) was that of an arch composed of wooden shear blocks. Great confidence was placed upon the shearing strength of these blocks, which were to have been cut in somewhat the same fashion as one finds in Japanese or Chinese frame construction. If memory serves, no such span as 3 500 ft. was contemplated by the designer. It is a coincidence, however, that one of the latest designs for the North River bridge crossing was also an arch; this time of reinforced concrete and for a span of 3 500 ft.! It is vouched for by at least two engineers, recently returned from abroad, that a French engineer, under the impression that the George Washington Bridge design would be competitive, prepared a design of a reinforced concrete arch for this span.

In earlier designs for the bridge under consideration there was little or no thought of a location so far north. It will be recalled that when the present streets and avenues were laid out for future development, little consideration was given to a future northward trend of traffic. It was supposed, with methods of transportation then in vogue, that traffic would be largely east or

[2] Prof. of Civ. Eng., New York Univ., New York, N. Y.

west or, from river to river. Hence, the congestion in the comparatively few north and south avenues in Manhattan. It is, therefore, highly significant that the first bridge joining New York City with municipalities in New Jersey should be located at 179th Street, roughly, a dozen miles north of Brooklyn Bridge, and not at Canal Street. In more than one instance, insistence by business people of a community to have through traffic routed so as to pass business houses, has proved to be a boomerang. Usually, considerable harm and little or no good has resulted from the ensuing traffic congestion. Thus, little or no tendency is found toward the development of a business section at the New Jersey end of the bridge, and on the New York side, a tunnel is actually in process of construction to move the major portion of bridge traffic away from the business section in 181st Street.

One phase of this paper deserves special attention. Engineers have acquired a rather unenviable reputation as being contemptuous of æsthetic feeling in their structural design. In days gone by, when it became necessary to cover structural elements, such as steel, with masonry, the covering was often not only without taste but out of harmony with the surroundings. The most beautiful engineering structures extant, Gothic cathedrals, are beautiful because the designing engineers (and they were engineers even though they were garbed in monk's gown and cowl) were so delighted with the perfected structural elements that they spent more thought and effort in emphasizing these than in any other parts of the structure. Thus, in true Gothic buildings, the structural elements are intentionally visible from both outside and inside. The first coverings of the modern steel skeleton buildings were hideous in appearance and out of harmony with the character of construction. Later methods practically begun with the Woolworth Building, emphasize the vertical as well as the horizontal structural lines and, behold, the beautiful skyscrapers! Editorial and other comment in the press and magazines during this period of development are eloquent on this point.

Designers for the Westchester County Park Commission in New York were also awake to possibilities of æsthetic appearance. Their rigid frame bridges, handsome in appearance to the engineer whose photo-elastic eye sees beautifully adjusted stresses, were made attractive and pleasing to the non-engineering eye by "fake" stone masonry covering. More than this, the engineers were very particular about the selection of local stone so that the character and color of the stone covering should be in harmony with surrounding landscapes. Mr. Ammann justly takes pride in the beauty of the design of the steel work in the towers of this bridge. Friends of the writer, critical of æsthetic effects, have expressed appreciation of the towers as they are. This pleasing appearance is particularly noticeable from the New Jersey side, when the towers are viewed with the tall Medical Center buildings as a background.

Provision has been made to cover the unethical appearance in concrete in the anchorage. It may be suggested that the stone cover be selected, as is done in Westchester parkways, to harmonize with the rocky cliffs of Washington Heights.

Masonry covering of prominent structures should be plausible. If it is to be carried by the tower steel, the towers must be relatively narrow. Narrow masonry towers of such height (600 ft.) may prove to be unconvincing to the layman's eye (say, from a river steamer), who sees the cables resting on them and not on the reliable steel bents. A tall and narrow masonry tower, apparently carrying such a heavy and shifting load, would appear to be very weak. This may defeat, effectively, the purpose of æsthetic treatment and prove to be just another attempt at "gilding the lily."

GUSTAV LINDENTHAL,[3] HON. M. AM. SOC. C. E. (by letter).—That part of Mr. Ammann's very valuable paper that deals with the historical account of the events and plans, preceding the conception and construction of the George Washington Bridge seems to require some elaboration and explanation as far as it relates to the writer and the North River Bridge Company. The author begins the chronology of the attempts to bridge the Hudson River with a reference to an Act of the New Jersey Legislature in 1868, for the New York and New Jersey Bridge Company, and he follows it with an account of that Company's plans before Congress, for a bridge with a pier in the river, and the subsequent reports by two Boards of Engineers, appointed by the Government in 1894, on the practicability of a single span over the river.

How this Company came to make any plans at all after an interval of twenty-six years is interesting, and should not be omitted in an accurate historical account of bridging the Hudson River at New York. For an explanation, it seems appropriate to give first the genesis of the North River Bridge Company mentioned by the author in second place. In the fall of 1885 the late Samuel Rea, Hon. M. Am. Soc. C. E., at that time Assistant to the Vice-President of the Pennsylvania Railroad Company, discussed with the writer, on behalf of his road, the practicability of a railroad bridge across the Hudson River. The writer was not the only engineer so consulted, as Mr. Rea was a very able engineer with a penetrating and cautious mind. He analyzed the situation to the writer as follows: There was keen competition among the railroad companies for Western traffic. The New York Central Railroad Company advertised a direct entrance, with four tracks, to the heart of Manhattan, while the Pennsylvania Railroad Company and the other railroads terminating in New Jersey were handicapped and had to transfer their passengers across the Hudson River by ferries. A tunnel under the Hudson River had been started at Hoboken, N. J., but it was intended only for small cars and local traffic. A larger tunnel for locomotives and standard cars appeared objectionable because of the smoke, which was then a subject of daily complaint in the tunnels of the New York Central Railroad. (Electric locomotives had not yet been invented.)

The great railroad bridge over the Firth of Forth in Scotland was then under construction. The question was, could a similar bridge be built over the Hudson River? A design for a railroad suspension bridge had just been discarded for the Firth of Forth Crossing, in favor of a rigid cantilever

[3] Pres. and Chf. Engr., North River Bridge Co.; Cons. Engr., Jersey City, N. J.

design. Evidently, a suspension structure, such as the Brooklyn Bridge or the Railroad Bridge over the Niagara River would not do. What was wanted was a rigid bridge for heavy locomotives and fast trains and with four tracks. This was the problem discussed in 1885. It was soon ascertained that the Government would not permit a pier in the river. In any case, the depth of rock (250 to 300 ft.) was prohibitive for piers. There must be a single span over the entire river, about 3 000 ft long. Could it be built?

The writer had given thought to the matter before, but he made further studies of the problem, and in the spring of 1886 presented a plan, and reported to the Pennsylvania Railroad Company that a rigid suspension bridge of four tracks and with a single span of 3 000 ft was practicable on a location near Desbrosses Street, New York City. The system for the trusses was that of suspended braced arches, known as the "garland" type, in which the chords were to be wire cables. The New Jersey approach was to be located over the Horsemus Cove Branch (owned by the Pennsylvania Railroad Company) opposite Desbrosses Street. The New York approach would turn north and descend to a terminal near Washington Square. The total cost for bridge and terminal would be about $22 000 000. This was a greater undertaking than the Pennsylvania Railroad Company at that time cared to assume. It was then considered that the bridge should be used by several railroads, that it should have six tracks, and should be located at 23d Street, with a terminal on Sixth Avenue, which was then the business center in Manhattan.

The North River Bridge Company was organized for that purpose in 1887, the writer was appointed Chief Engineer, and, in 1888, application was made to Congress for a franchise. The plans were explained to Gen. Lincoln N. Casey, then Chief of Engineers, U. S. Army, who recognized them as practicable. The charter was granted in July, 1890. At the behest of the local members in Congress who desired rapid transit over the bridge, the number of tracks prescribed in the Act was increased from six to not less than ten.

The suspension design for the 23d Street Bridge location (published in 1888) was the first plan for a bridge over the Hudson River at New York, and it was for a rigid railroad bridge at that. All other designs were proposed by later engineers, a fact which appears to be obscured in the author's introduction. The feature of a very long suspended span was at no time the problem—this was always simple enough since it requires merely heavier cables—but the construction of a suspension bridge of long span, sufficiently rigid and economical for heavy railroad traffic at high speeds on several tracks was the actual problem, for the solution of which the writer proposed his design of the garland type. He is still convinced that it is the most economical of all types for that purpose. The feasibility of a near rigid suspension bridge for railroad loads and for a span of 3 200 ft without stiffening frame of any kind, but also its prohibitive cost, was discussed by the writer in his paper on "A Rational Form of Stiffened Suspension Bridge."[4]

The location of a bridge across the Hudson River at 23d Street was approved by the War Department in December, 1891, and work was commenced in 1892; but no sooner did the plans of the North River Bridge

[4] *Transactions*, Am. Soc. C. E., Vol. LV (1905), p. 64.

Company become known, than a rival bridge company sprang up with an old New Jersey charter, granted in the speculative era after the Civil War. Its scope and jurisdiction were supplemented with legislation in Albany, N. Y., in 1890. This Company was The New York and New Jersey Bridge Company described by the author in his introduction. It was advised by its engineers that the plans of the North River Bridge Company were impracticable for railroad purposes.

The argument quoted by Mr. Ammann against "spanning the North River without a pier,"[2] voiced the opinion then prevailing among leading bridge engineers, and was directed against the writer's plans. A rigid cantilever bridge with piers in the river was proposed, but the plan was opposed by the War Department. It led to the appointment of two separate Boards of Engineers by the Government, to investigate the practicability of a single span for six railroad tracks. By that time the construction of such a bridge had been authorized by the Government, and had actually been started on plans by the writer; but it was interrupted by litigation, to which the New York and New Jersey Bridge Company was a party, disputing the question of the right of condemnation of land claimed by the North River Bridge Company. It was decided by the U. S. Supreme Court in 1894 in favor of the North River Bridge Company. The reports of the two Boards of Engineers in 1894 are quoted by Mr. Ammann. Both reports discuss the plans of the writer, and one of these reports contains a detailed description of the North River Bridge of that time. All this happened in 1894, several years after the writer's plans were first publicly known.

The plans of a railroad bridge over the Hudson River, unlike those of a bridge for only highway or rapid transit, are inseparable from plans for railroad terminals and connections; therefore, the plans for the 23d Street Bridge included a terminal station on Sixth Avenue, and a rail connection with the Long Island Railroad, through the so-called Steinway Tunnel under 42d Street, which was then being built. Had the railroads combined with the Pennsylvania Railroad Company in 1890 to build the 23d Street Bridge, it would have caused an estimated addition of 1 500 000 population to Northern New Jersey by this time.

Subsequent events and a business depression, as related by the author, caused delays and led to changes in the plans. The exigencies of the World War intensified and congested traffic on the river and in the harbor. An exceptionally cold winter froze the ferry slips, and for an entire week interrupted the transfer of car-floats carrying food and fuel to Manhattan. The public demand for a bridge became more urgent than ever. Thereupon the North River Bridge Company, in consultation with the Chief Engineers of all the railroad companies (nine in New Jersey and three in New York) prepared the plans for a large freight terminal, which would be directly accessible to ocean shipping and from thirty intersecting streets in the warehouse district. The plans included also a large passenger station on Eighth Avenue.

The bridge was only about one-third of the entire project, which was judged to be the best conceived for its purpose. It had been developed in

detail with the aid of railroad transportation experts. The combined traffic would require twelve tracks over the bridge (four for freight, four for passenger trains, and four for rapid transit), all on one level, which explains the cross-section (Fig. 4) given in the author's paper of the double-deck floor-beam, arranged on the Viereendel system. Highway traffic would find room as needed on the upper deck.

More recently, revised plans for the 57th Street Bridge and terminals, to meet the changed conditions, have been prepared. It is proposed to build the bridge in several stages, adjusted to the exigencies of transportation as they may develop for a roadway and heavy railroad structure, intended to endure many centuries.

HAROLD M. LEWIS,[5] M. AM. SOC. C. E. (by letter).—The George Washington Bridge over the Hudson River, as described in Mr. Ammann's paper, appeals to many because of its tremendous size and the physical difficulties overcome in its construction. It is also of great importance in its probable effect on the pattern of the future highway system of the New York Region.

Place in Regional Plan of Communications.—Earlier projects for a Hudson River Bridge at New York City were put forward primarily to provide a new connection between New Jersey and the metropolitan business centers on the Island of Manhattan. Public agitation for new transportation facilities is generally directed toward those that parallel already congested routes in the same vicinity. As a part of the regional highway system, the George Washington Bridge is essentially a way around the congested centers and, to this extent, it provides an entirely new facility. The Port of New York Authority, with its independent method of financing its projects, has been able to locate this crossing where it is most needed to improve the distribution of vehicle traffic in the entire Metropolitan District rather than where it would serve merely as another artery to encourage the continuance of a poor distribution. It is the major link in the proposed metropolitan loop highway in the Regional Plan.

Importance of Rail Facilities.—The George Washington Bridge eventually should also become part of a transportation "corridor" around the central business and residential areas in the Metropolitan District. Such a "corridor" will provide, in places, for all types of ground transportation. From a planning point of view, to let rail facilities from New Jersey terminate in Northern Manhattan and compel transfer to the limited local transit lines in that part of the city, would be as faulty as to terminate the highway approaches there.

From its early studies the Staff of the Regional Plan of New York and Its Environs has considered a bridge at the site of the present one as an important link in the railroad, as well as in the highway, system of the Region. The importance of considering rail facilities in the design of the bridge was pointed out by the writer on behalf of the Regional Plan at a meeting of the New York and New Jersey Hudson River Bridge Advisory Committee of the Port of New York Authority on March 12, 1926.

[5] Cons. Engr.; Engr., Regional Plan Assoc., Inc., New York, N. Y.

At that time the writer emphasized the fact that a bridge at 179th Street would serve not only as immediate relief to traffic conditions, but would furnish an incentive for, and make possible the development of, any decentralization that seems desirable. Studies by the Regional Plan of New York and Its Environs had indicated that, in order to serve adequately in this way, the bridge and its connections should be designed so that eventually it could carry every possible kind of transportation that may flow over it in the future, including pedestrians, trackless vehicles, and railroad services of all kinds.

These recommendations were based upon a plan for regional rail facilities, which had been worked out under the guidance of William J. Wilgus, M. Am. Soc. C. E., who acted as Consultant on studies of transportation and port development for the Regional Plan. His recommendations for rail links between the bridge and both the New Jersey and New York railroads are incorporated in the Graphic Regional Plan.

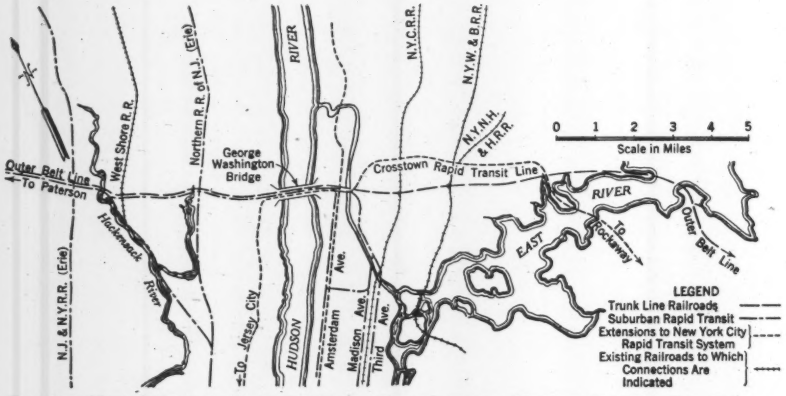


FIG. 1.—MAP SHOWING POSSIBLE RAIL USES OF THE GEORGE WASHINGTON BRIDGE OVER THE HUDSON RIVER.

The Regional Plan gave consideration to three kinds of rail transportation: Trunk-line railroads, suburban rapid transit, and local rapid transit. The way in which the George Washington Bridge might serve these three systems of rail transportation is shown in the writer's Fig. 1. It is realized that the use of the bridge for rail purposes involves the co-operation not only of the railroad companies, but also of many public agencies. As explained in Mr. Ammann's paper, the Port Authority Staff felt that "the prospective volume of [rail] traffic fully warranted the comparatively small expenditure which was necessary to provide for four rapid transit tracks on the bridge," which can be accommodated on a future lower deck. Even though it may be many years before rails are actually added to the bridge, it is not too soon to develop plans for suitable connections with them. The proposal to carry local rapid transit tracks across the bridge to link Hudson County with Manhattan may seem to some to be beyond the realms of possibility,

due to political difficulties resulting from the State line in the center of the river. Perhaps the time may come when practical advantages may be given greater consideration than political expediency.

Estimates of Future Traffic.—The forecasting of future traffic on a structure which provides a new type of facility is a most difficult problem. Past trends are insufficient, as new traffic may be created by the new facility. The Regional Plan attempted to create a graphic picture of future vehicular traffic in the New York Region by the "distribution method." This was based upon sub-dividing the Region into a certain number of districts, each of which would have the same future population, and estimating the future traffic between each of these districts and every other district. The amount of traffic was considered a function of registration and distance. Based on a population of 21 000 000 in the Region in 1965, the vehicular traffic in that year between Manhattan and New Jersey was estimated at about 55 000 000. From this and past records up to and including the year 1926, estimates were prepared in 1927 for future trends in both the total traffic between Manhattan and New Jersey, and those parts of it which would be north and south, respectively, of 40th Street in Manhattan. These estimates up to 1950 are shown in the writer's Fig. 2 where they are compared with

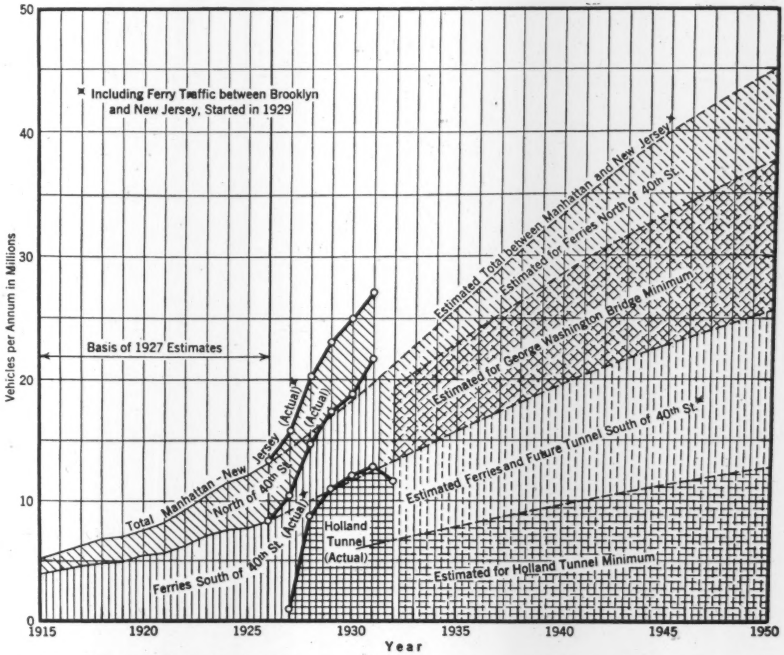


FIG. 2.—VEHICLE TRAFFIC ACROSS THE HUDSON RIVER AT NEW YORK CITY, 1915–1950. ESTIMATES MADE IN 1927 COMPARED WITH ACTUAL TRAFFIC, 1927–1931.

what has actually happened since 1926. The diagram also includes estimates prepared in 1927 for the minimum traffic through the Holland Tunnel and across the George Washington Bridge.

It appears in the diagram that, in 1931, traffic in the Holland Tunnel of 12 756 193 vehicles was about twice the minimum estimated by the Regional Plan. The new traffic created by this facility had shoved the curves of actual traffic well above the estimated curves both for the total between Manhattan and New Jersey and for those crossing the Hudson River south of 40th Street, Manhattan. During its fifth year of operation ending November 13, 1932, the Holland Tunnel carried 11 636 446 vehicles. This has been added to Fig. 2 and shows an 8.8% loss over 1931. The downward trend is undoubtedly only temporary, but has certainly held down the traffic over the George Washington Bridge below that which would have occurred normally. Nevertheless, this traffic during the first year of operation of the bridge, ending October 15, 1932, and amounting to 5 628 234 vehicles, is fairly close to the Regional Plan estimate of 6 100 000 vehicles in 1932.

The writer believes that the 1950 estimates for total traffic between Manhattan and New Jersey and that over the George Washington Bridge, as shown in Fig. 2, still present conservative figures for that date. They might be considered as minimum estimates which are likely to be exceeded, as is indicated by the estimates in Mr. Ammann's paper. The latter also include the Yonkers, Piermont, and Tarrytown ferries, which are omitted in the estimates in the accompanying diagram.

O. H. AMMANN,[6] M. AM. SOC. C. E. (by letter).—The writer appreciates very much the valuable contributions and the complimentary comments on his paper. The excellent exposé of the fundamentals in bridge design by Mr. Howard, and his recognition of their application to the George Washington Bridge are gratifying. Undoubtedly, like the writer, Mr. Howard has struggled to extricate himself from the maze of structural conceptions which again and again appear to grip the imagination of the engineer and lead him to devise complicated, and sometimes abortive, solutions.

Time is an important factor in the mature development of the design of an important structure, but, unfortunately sufficient time is often lacking. Structures have been put on paper hurriedly, sometimes not even to scale, and from that point constructed with amazing speed in the field, with no thought of æsthetic conception or refinement in design. It is to be hoped that the era of driving prosperity will have taught a lesson in this respect and that in the future engineers will be able to proceed more deliberately in the conception and moulding of important structures.

Mr. Howard wonders where 50 000 000 more vehicles to cross the Hudson River annually will operate in Manhattan. This figure appears impressive, but when this is compared with the number of annual vehicle trips to and from Manhattan across the Harlem and East Rivers, which is more than

[6] Chf. Engr., The Port of New York Authority, New York, N. Y.

200 000 000, and with the still vastly greater number of vehicle trips localized in Manhattan, the figure becomes almost insignificant.

Moreover, unlike the commuter traffic, which pours into Manhattan during a few hours in the morning and leaves it similarly in the evening, the vehicular traffic across the Hudson River is more uniformly distributed, and almost throughout the day one vehicle leaves Manhattan to every one that comes in. Additional Hudson River crossings tend to increase traffic circulation; they not only impose new traffic on the streets of Manhattan, but they also relieve traffic "bottled up" there.

The increase in number of vehicles across the Hudson River will come not so much from increase in population, as Mr. Howard assumes, but by far the greater proportion will come from a spread of population to the suburbs and from encouragement of more frequent use of motor vehicles, both induced by better highway facilities. These facilities for traffic in and out of Manhattan in all directions are yet far from adequate and economical to meet the demands when normalcy returns. This applies particularly to through, or long-distance, traffic which, while yet relatively unimportant, as Mr. Howard properly recognizes, is bound to assume voluminous proportions in course of time.

Mr. Howard's recognition of the importance of adequate highway approaches cannot be appreciated too strongly. Developments in the future will justify even more elaborate and advanced arrangements than those that had to be adopted for the George Washington Bridge under prevailing conditions.

The writer is encouraged by the emphasis laid upon the æsthetic side of bridge design by Professor Schwarze and his acknowledgement of the efforts in this respect in the case of the George Washington Bridge. Conditions, more particularly the naturally imposed length and proportions of spans and their relation to topography, were not conducive to rendering this bridge a structure of outstanding beauty. Whatever æsthetic merits the George Washington Bridge possesses are, in the writer's opinion, due largely to the structural simplicity and functional clarity of the structure as a whole. The design of the towers, which has called forth varied views and comments is of secondary æsthetic effect. Undue weight has been attached to it in some criticisms. As may now be fully appreciated by viewing the bridge from a distance, in its setting in the landscape, the proposed covering of the steel skeleton with masonry would scarcely alter the picture and the writer, judging from the architect's perspectives, does not share the concern of Professor Schwarze that such encased towers might appear too slender and weak. The outside proportions of the towers, whether they were to remain bare or were to be encased, were purposely selected with a view to expressing their function of carrying an enormous load, but without producing undue massiveness. Preference for the steel towers as they stand, or for encased towers as originally proposed, is entirely one of individual taste.

The writer is indebted to Mr. Lindenthal for his very interesting and valuable contributions to the history of bridging the Hudson River with which he has been so intimately connected and which centers itself largely around his own untiring efforts.

Mr. Lewis calls particular attention to the suitability of the George Washington Bridge for railroad services in accordance with the early studies embodied in the Regional Plan of the City of New York and Its Environs. The writer is in entire sympathy with this idea and provisions in the design of the bridge will make it feasible to carry it out, at least to the extent of providing four, possibly six, rapid transit tracks. The extent to which bus passenger traffic over this bridge is constantly increasing in volume is evidence of a demand for this service, which eventually, however, will assume proportions that cannot be served effectively except by rapid transit.

The diagram of vehicular traffic across the Hudson River presented by Mr. Lewis (Fig. 2) illustrates a phenomenon that has been manifested prior to the present economic depression in almost every instance of a new transportation artery in and around New York City, namely, that estimates of traffic volume remained far behind actual developments. The Port Authority Staff has had to increase its forecast estimates of Hudson River traffic repeatedly. The revision made in 1930, based upon traffic trends at that time, is illustrated in Fig. 8 of the writer's paper. The depression has put a stop to the accelerating increase and its effect will undoubtedly be felt for many years to come. A review of its estimates in September, 1932, has led the Port Authority Staff to forecast traffic that is between the rate shown in Fig. 8 and that in Fig. 2 of Mr. Lewis' discussion. Fig. 3 was prepared in Septem-

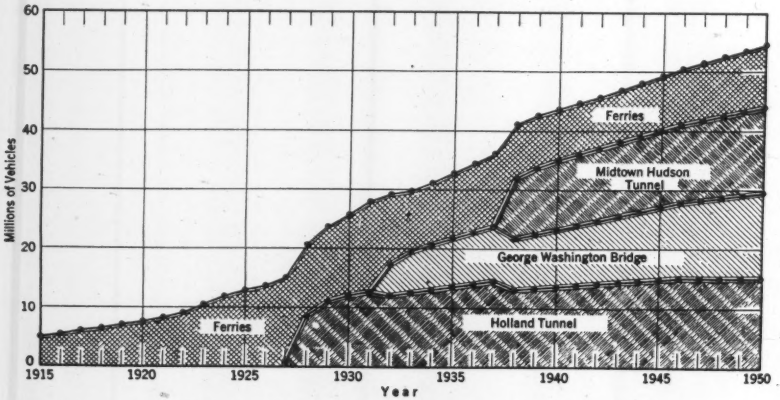


FIG. 3.—RECORDED AND ESTIMATED ANNUAL VEHICULAR TRAFFIC FOR ALL HUDSON RIVER CROSSINGS.

ber, 1932, and should be compared with Fig. 8 of the writer's paper which embodies the traffic estimates made by the Port Authority Staff in 1930. The traffic data represented in Fig. 3 formed the basis of the application of the Port Authority to the Reconstruction Finance Corporation for financing the Midtown Hudson Tunnel at 38th Street, Manhattan, as a self-liquidating project under the Emergency Relief Construction Act of 1932.

The writer agrees with Mr. Lewis that the Regional Plan forecast for 1950 is still conservative and may be regarded as a minimum estimate. The traffic over the George Washington Bridge, as actually developed in 1932, practically agrees with Mr. Lewis' forecast of 6 100 000 vehicles.

ORGANIZATION, CONSTRUCTION PROCEDURE, AND CONTRACT PROVISIONS

Discussion by Messrs. J. P. Carlin, and Edward W. Stearns

J. P. Carlin,[7] M. Am. Soc. C. E. (by letter).—As a principal in two of the contracts in connection with the George Washington Bridge, with its resulting association with the various divisions of the Port Authority engineering organizations, the writer feels, after carefully reading Mr. Stearn's paper, that nothing can be added thereto without going into details, apparently omitted for the sake of brevity.

A part of the paper under "Form of Contract" relates to a subject of great importance in construction work where adjoining properties might be affected by contract operations. Mr. Stearns states that full responsibility was placed upon the contractor for his work and for all damages and claims for damages resulting from it. He states also that the contractor was the insurer of the Port Authority against all contingencies arising out of the performance of the work, and that this course was justified because of the thorough studies, surveys, and sub-surface investigations that preceded the issuing of the contract documents, and because the complete and detailed drawings and specifications reduced to a minimum the uncertainties faced by the contractor.

Frequently property owners take advantage of this blanket indemnity running in favor of the other party to the contract, and make claims against the contractor for every conceivable damage that may have occurred to their property during the course of contract operations. Recently, a particularly voracious owner presented claims for repairs alleged to have been caused by a contractor's operations, whereas the repairs were made the year prior to the signing of the contract. Another case was that of a tenant in one of the apartment houses adjacent to a job, who complained that blasting operations had damaged his radio set; whereas there were twenty-five other tenants in the same building, none of whom made a similar claim, and all of whom had radio sets. Frequently, the property owner will claim damages to a sidewalk at points as much as 100 ft. away from the work, where construction operations could not possibly have affected it, and where observation disclosed that coal trucks and moving vans had actually caused the damage. Still another claimed damages to his roof, alleging that it was caused by the contractor's operations, and upon investigation it was found that the roof had been placed sixteen years previously, and that the galvanized gutters and flashings were completely corroded. All these cases confirm

[7] Pres., The J. P. Carlin Constr. Co., New York, N. Y.

a statement[8] made by Miles I. Killmer, M. Am. Soc. C. E., who points out that in many instances some owners will seize upon the occasion of the presence of a contractor to bring suit even when the physical damages are so slight as to require magnifying glasses to be seen.

These instances illustrate the importance of making a preliminary examination, and show that the contract should require such a survey to be made of all houses and property along the line, or adjacent to the proposed construction. If included in the specifications of the proposed work, it would eliminate all doubt as to the obligation of the owner of the improvement to establish a record of the existing conditions prior to the commencement of the work by the contractor, and to which record the property owner should be obliged to subscribe, together with the successful contractor.

Another interesting point is the matter of liquidated damages for failure to complete the work on time, which clause has quite generally displaced the bonus and penalty agreement which was common at or about the time of the World War. If the owner of an improvement is injured by its delay in completion, it follows that he must benefit if the work is completed earlier than the contract time. Furthermore, a public improvement is an economic necessity and the sooner it is completed, the quicker it will function and fulfill the necessities of the public. If, therefore, both the owner and the public are benefited by earlier completion, an incentive should be offered to the contractor to partake of a portion of this benefit which, in effect, would be a part of the owner's carrying charges, and which would otherwise be wasted.

It is erroneous to assume that the entire bonus for earlier completion represents profit, because the contractor spends a part of this bonus for the purchase of additional equipment, and absorbs the resulting inefficiency which follows from speeding up the work. The bonus clause is of great advantage to the owner if the work is properly co-ordinated because it assures earlier completion than by any other clause, such as that of liquidated damages.

Where several independent contracts are involved, as was the case on the George Washington Bridge, the writer believes, with Mr. Stearns, that a bonus arrangement would not have redounded to the advantage of the Port of New York Authority. It is rare, however, that such perfect co-ordination of all the reciprocating parts of an enterprise of the magnitude of the George Washington Bridge can be as nicely calculated, and anticipated, as was the case on this bridge. The result is an outstanding tribute to the skill and efficiency of the engineering organization of the Port of New York Authority, and it evidences co-operation on the part of all the contractors engaged on this work.

EDWARD W. STEARNS,[9] M. AM. SOC. C. E. (by letter)—The writer appreciates greatly the interesting communication from Mr. Carlin, although he cannot agree with him in all his comments. Mr. Carlin's remarks have especial reference to the contract for the approach tunnel under 178th Street, Manhattan, where abutting properties were subject to damage by the conduct

[8] *Proceedings*, Am. Soc. C. E., October, 1932, p. 1392.
[9] Asst. Chf. Engr., The Port of New York Authority, New York, N. Y.

of the construction operations. The writer is entirely in accord with the importance of a preliminary examination of such properties and believes that, imperfect as such a proceeding is, it is, nevertheless, if properly done, the one best way to establish the extent of damages caused by the work. He believes, however, that inasmuch as such a determination is of first importance to the contractor, the undertaking of such an examination should originate with the contractor himself and not with the owner.

The advisability of including a provision in the contract for the approach tunnel under 178th Street requiring such preliminary examination was seriously considered, but such provision was believed to be superfluous in view of the clause in the contract that provides that "the contractor shall do all things necessary or proper for or incidental to the work," in addition to the provision that "the contractor shall be the insurer of the Port Authority against the work of damage to the property of third persons, arising out of or in connection with the performance of the work." If, as Mr. Carlin indicates by his comments, the contractor considered a preliminary examination necessary and proper, then, by the terms of the contract, such an examination was required as part of his contractual duties. The Port Authority holds that such an inspection is primarily in the interest of the contractor himself and if included as a requirement of the contract, would tend at least to make the Port Authority jointly responsible with the contractor.

Mr. Carlin makes the general statement that "if the owner of an improvement is injured by its delay in completion, it follows that he must benefit if the work is completed earlier than the contract time." This statement would be quite axiomatic if it were not a fact that in most undertakings, and particularly so in public work, there are usually external conditions over which the owner has no control and which are likely to make it impossible to reap the benefit of earlier completion. The writer is very glad that Mr. Carlin agrees with him that a "penalty and bonus" provision in its contracts would not have benefited the Port Authority. The proper co-ordination of the various elements affecting the completion of an undertaking is a problem, the complexity of which varies more or less directly with the magnitude and complexity of the undertaking itself, and is just as much a real problem for the engineer to solve as are the problems of locating the piers or determining the sizes of the various members. An engineer is remiss in his duty if he fails to give due consideration to all these elements and then inserts a penalty and bonus provision in his contract to cover up his laxity. In the writer's opinion, it is far better to take account of the conditions which must be met, arrive at a reasonable and logical time for the work to be completed, and then draw a contract which will either lead to its completion within that time or reimburse the owner for any loss which he may suffer by reason of the lateness of its completion.

It would seem as if there were little difference in dollars and cents between the two systems under the competition system of bidding. If the time set up is ample, the contractor is likely, under the bonus system, to predetermine

his probable bonus and then deduct that amount from his bid price, whereas under the system of liquidated damages he will estimate the most economical time for him to consume in doing the work, and his bid price will reflect the resultant economy. If, on the other hand, the time set up is very limited, so limited in fact as to make it difficult or impossible to complete the work on time, the contractor will increase the amount of his bid price so as to be reimbursed either for too rapid work or for the penalty imposed for failure to complete on time, or for both.

CONSTRUCTION OF SUBSTRUCTURE

DISCUSSION BY EDWARD P. PALMER, M. AM. SOC. C. E.

EDWARD P. PALMER,[10] M. AM. SOC. C. E. (by letter).—Referring to that part of the paper by Mr. Case dealing with the use of continuous belts for transporting concrete materials for the New York anchorage it may be of interest to review some considerations which led to the choice of that method. While the horizontal distance from the unloading dock to the mixers was only 1 000 ft, the ground level at the mixers was 100 ft higher than at the unloading dock, and the top of the storage bins 60 ft above the ground, making the vertical distance which the material was to be transported 160 ft, or about one-sixth as much as the horizontal distance.

The alternate method of transportation considered was the use of trucks for the horizontal distance and the first 100 vertical ft, and a bucket conveyor for the additional 60 vertical ft. It was found possible to lay out a trucking road about 1 300 ft long, with an average gradient of about 7½ per cent. To supply the required material it was estimated that 30 loads per hr would have to be hauled. This required a two-way road, and the 60 crossings per hr over the New York Central Railroad tracks made it appear that it would be necessary to construct a bridge at this point. Owing to the contours, a considerable part of this trucking road, in addition to the bridge, would have to be built on a trestle. Consideration was given to the cost of maintaining such a structure, and, also, in view of the fact that the trucking would be done during the winter months, to the possible difficulties resulting from icy weather.

All these considerations pointed to the choice which was made. The wider sand and gravel belt was placed over the cement belt, giving the cement sufficient protection except during rare, driving rain storms.

Mr. Case also describes the method of distributing mixed concrete; that is, by means of a belt from the mixers to the base of the tower from which concrete was distributed through chutes in the usual manner. It is here that the writer thinks the plant would have been more efficient had belt conveyors been substituted for chutes. While large chutes were used, and every

[10] Secy. and Treas., Senior & Palmer, Inc., New York, N. Y.

care was taken to obtain uniformity of mix and a consistency suitable for chuting, the chutes at times were plugged. It appears entirely feasible to design a plant with belt conveyors suspended somewhat in the same manner as the chutes. It is believed that, on account of the low cost of operation, such conveyors would have more than paid for the initial cost through the savings in the height of the tower and in the elimination of lost time necessary to clear the chutes.

APPROACHES AND HIGHWAY CONNECTIONS

DISCUSSION BY MESSRS. S. WOOD MCCLAVE, JR., WILBUR J. WATSON, HAROLD M. LEWIS, HUNLEY ABBOTT, AND J. C. EVANS

S. WOOD MCCLAVE, JR.,[11] M. AM. SOC. C. E. (by letter).—The subject of approaches and highway connections to the George Washington Bridge has been admirably covered in the paper by Mr. Evans. When the New York approach is finished it will be as nearly perfect as can be accomplished by engineers who are confronted with the many difficulties arising from any closely built-up section.

The New Jersey approach, in the Borough of Fort Lee, presented an extremely difficult situation from the municipal standpoint. The original plan of the Port of New York Authority had been approved by the Mayor and Council to the easterly side of Lemoine Avenue and a circular plaza was suggested at Lemoine Avenue because, at that time, no State Highway plans had been made for the traffic expected from the west. The Highway Department had suggested a proposed plan that was not satisfactory to the Borough.

Representatives of the New Jersey State Highway Commission, Bergen County, and the Borough of Fort Lee, in New Jersey, were appointed by the respective bodies to serve with the Port Authority representative (the Chief Engineer) on an Engineering Committee. After two years of the most difficult kind of work this Committee decided on the present plan, which was immediately accepted by all the governing bodies represented by their engineer.

This approach, with its marginal roads, eliminates the possibility of seeing any backyards from the main highway. It also enhances in value the property immediately facing the approach. It serves the purpose, with right-hand turns only, of taking care of all local traffic, while through traffic can proceed with safety at high speeds without the aid of a traffic light or traffic police. There are no grade crossings to interfere with the traffic, and, consequently, there is no interference with fire protection. This approach should serve as a model for all bridges in the future.

WILBUR J. WATSON,[12] M. AM. SOC. C. E. (by letter).—An important definition of a bridge approach is contained in the paper by Mr. Evans, under the heading, "Approaches as Major Elements of the Bridge Project." It is inter-

[11] Civ. and Cons. Engr. (McClave & McClave), Cliffside, N. J.
[12] Archt. and Engr., Cleveland, Ohio.

esting in this connection to note that the problem of what constitutes a bridge approach actually led to litigation in connection with the Lorain-Carnegie Bridge, in Cleveland, Ohio, built under the auspices of Cuyahoga County.

Before work on the design was begun, the City and County authorities came to a verbal agreement as to the limit to which the County would go in building the approach structure, and it was understood that the City of Cleveland would continue the work from that point. As in the case of the Port of New York Authority and the Highway Department of the State of New Jersey, the broad plan involved the widening and opening of streets several miles from the bridge site, and, consequently, there was no definite limit as to what constituted the approach.

As the work neared completion, it became evident that the City could not abide by its part of the verbal agreement, for financial reasons; but the bridge proper was built for about three-fourths of the estimated cost, mostly because of decreased costs of labor and materials; and, therefore, the County authorities proposed to undertake to complete the approaches as least as far as the available funds would permit.

Legal advisers for the County, however, ruled that money from the authorized bond issue could not be used for this purpose inasmuch as the work contemplated could not properly be included as the bridge approach, the wording of the bond issue being "for the bridge and the necessary approaches thereto." It was finally taken to the Courts in a friendly suit, to determine the meaning of the word "approach."

A diligent search of existing literature indicated that there was no clear and adequate definition of the word other than, for example, "the construction leading to the end of a bridge." Accordingly, the writer offered the following definition to the Court, after certain necessary legal phraseology had been added by the attorneys:

"The approaches to a bridge comprise the traffic arteries leading to the ends of the bridge proper, and such adjustments of alignments and grades of said arteries in the immediate vicinity of such ends as is necessary to afford the maximum convenience of access, and render available to the public the entire capacity of the bridge proper."

As a part of the preliminary studies for the bridge, E. J. McIlraith, Traffic Engineer, of Chicago, Ill., submitted a report in which was contained a remarkably clear statement of the basic principles underlying the design of approaches, as follows:

"Since bridges are necessarily limited in number each becomes of great importance as an artery of travel and the use made of it is dependent on the convenience of the approaches. It is also vital that the streams of traffic flowing to and from a bridge do not cause harmful congestion to business located near the bridge terminal. The arrangements should be such as to spread benefits to all rather than to create damage to some.

* * * *

"The major consideration should be to develop a definite city planning approach which considers the effect of a proposed construction on the increase

in usefulness and value of the affected sections, and provides necessary and desirable thoroughfares, routings, and betterments for traffic that will permit and encourage great improvements in freedom of traffic flow. Traffic studies indicate supplementary information to show when each of several probable bridges will become necessary, but the exact location of each bridge and the treatment of its approaches must be based on effective street planning that will serve as part of the ultimate plan for the city's greatest possible growth.

"The general scheme of bridge location and of development of the approaches to the bridge must therefore be set up boldly to fit an outstanding need and in the interest of the greatest civic good."

A comparison of the wording of the report with the definition given by Mr. Evans for the approaches to the George Washington Bridge, shows a remarkable agreement between the two.

HAROLD M. LEWIS,[18] M. AM. SOC. C. E. (by letter).—In the description of the proposed regional highway system of the Regional Plan of New York and Its Environs, the George Washington Bridge is referred to as the "keystone" of a metropolitan loop highway encircling the central areas of the Region about fourteen miles from New York City Hall. It is on the northerly section of such a loop, which eventually would re-cross the Hudson River at its mouth at The Narrows. This loop has been laid out to connect all the main radial highways in the outer areas, and over sections of it vehicles may gain access to the most direct route to their destination in the central areas. Through traffic from all directions will find it a convenient by-pass around the congested districts. The loop crosses the Arthur Kill by the Goethal's Bridge, another Port Authority structure.

The George Washington Bridge will be used more and more as a by-pass for traffic going through New York City on its way between New England and the Middle Atlantic States, or for traffic between The Bronx, Queens, Brooklyn, or Northern Manhattan, on the one hand, and New Jersey or points west or northwest, thereof, on the other hand. Therefore, the approaches to the bridge are of the utmost importance and must be connected with all radiating trunk highways on both sides of the river.

The State of New Jersey has provided such connections in an admirable way with its system of State highways radiating to the north (Route 1); the northwest (Route 2); the west (Routes 4 and 6); and the south (another section of Route 1). Within ten miles of the George Washington Bridge, the New Jersey State Highway Department is spending approximately $40 000 000. Of course, the highways involved will supply future needs for much traffic that does not cross the bridge, but their construction up to the present time (1933) has been promoted primarily because of the presence of the bridge. The State Highway Engineer has estimated that about one-half this total cost is chargeable to bridge traffic.

What is being done on the east, or New York, side? No adequate provision has been made for connecting with trunk-line highways leading to the east or northeast. Of course, the problem was much more difficult on the New York City side of the bridge because adjoining areas were built up and

[18] Cons. Engr.; Engr., Regional Plan Assoc., Inc., New York, N. Y.

land was many times as expensive. The Port of New York Authority has undertaken to bring a double vehicular tunnel as far east as Amsterdam Avenue in Manhattan, where New York City has projected a serpentine series of connecting roadways within the boundaries of Highbridge Park.

It is just as important to have main highway approaches, free from traffic interruption, connect with the routes to New England, The Bronx, and Long Island, as it is to provide similar connections in New Jersey. In spite of the difficulties from interference with existing developments, adequate future connections, if planned now, should not be impossible. Looking ahead to the completion of the full potential roadway capacity of the George Washington Bridge and the future addition of rail facilities on its lower deck, some provision must be made for extending such facilities to the east by new connections.

A bridge across the Harlem River designed to carry both rails and vehicular roadways provides a logical solution for this problem. The Regional Plan included such a proposed bridge between West 178th and West 179th Streets and a new bridge at this site has been considered by the Department of Plant and Structures of the City of New York and endorsed by local civic organizations.

The original study by the Regional Plan was prepared in 1926. It called for a two-level bridge across the Harlem River with a roadway on the upper deck and provision for future rail facilities on the lower deck. In Manhattan it proposed a depressed roadway in open cut on the north side of West 178th Street, passing under all the avenues east of the George Washington Bridge Plaza. On The Bronx side the bridge roadway connected with University Avenue, the main roadway continuing partly in open cut and partly in tunnel to join East 170th Street at the foot of the ridge between the Harlem River and Jerome Avenue.

Mr. Evans has stated that such a plan for the Manhattan approaches between the George Washington Bridge and Amsterdam Avenue was considered by the Port Authority, but was abandoned due to the amount of property required and to the objection by the City authorities to the destruction of so much property value. The approach plans that were adopted appear to be designed for a traffic movement that will be altered materially when a new Harlem River Bridge is constructed. The efficiency of a temporary arrangement is not as important as that of the final plan.

The Regional Plan proposal for a Harlem River Bridge has been revised to adapt it to the plan for vehicular tunnels under West 178th and West 179th Streets, as adopted by the Port Authority and the City. It is shown in the writer's Fig. 4, on which heavy lines indicate new facilities in addition to those already under construction.

The axis of the proposed Harlem River Bridge would intersect Amsterdam Avenue at the center line of West 178th Street, where a connection would be made with the eastbound tunnel (to be operated temporarily as a two-way tunnel) in that street and the projected future westbound tunnel in West 179th Street. All traffic using these tunnels as a route between The Bronx and the George Washington Bridge would also use the new Harlem River

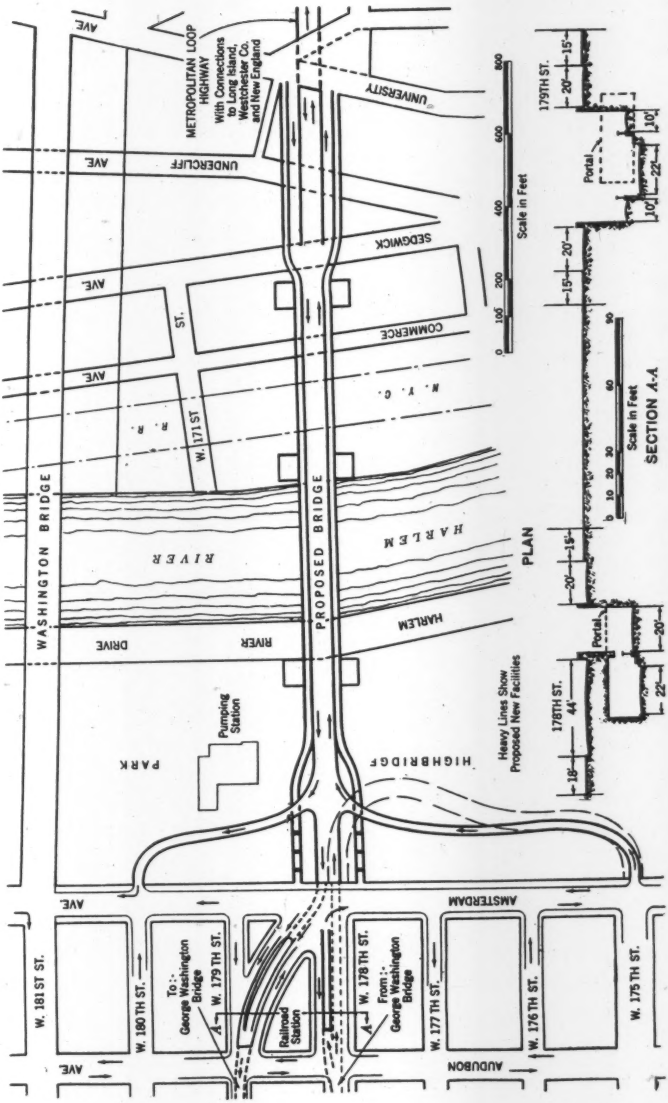


FIG. 4.—STUDY FOR AN EASTERLY EXTENSION OF THE GEORGE WASHINGTON BRIDGE APPROACHES AS PART OF A METROPOLITAN LOOP HIGHWAY.

Bridge. Amsterdam Avenue would become an important approach to the George Washington Bridge from the main part of the city to the south.

An exit ramp from the eastbound tunnel is indicated on the north side of West 178th Street, which would permit traffic using it to make a right-hand turn south into Amsterdam Avenue. It is believed that traffic desiring to leave the tunnel at this point could be separated safely from the two lanes of moving vehicles because the small amount of crossing involves only one lane and a movement that is carried out without difficulty or hazard at many points of a similar nature above ground. An entrance from Amsterdam Avenue to the westbound tunnel would be provided by two ramps near 179th Street, one of these for trucks and the other for passenger vehicles avoiding, in this case, any crossing of traffic lanes and providing full visibility for drivers of vehicles on the ramps or in the main tunnel.

As Amsterdam Avenue is about 15 ft lower than Audubon Avenue in this vicinity, these connecting ramps, as shown on the plan (Fig. 4), will readily fit in with maximum grades of 4 per cent. On The Bronx side the highway connections with University Avenue would be approximately level, while the main roadway would pass under University Avenue on a 4% grade. The roadway on the proposed Harlem River Bridge would be at an elevation of 140 ft above mean high water. With provision for future railroad tracks on a lower deck and an arch structure similar to that on Washington Bridge and the rebuilt section of High Bridge, there would be a clearance of about 110 ft over the center of the waterway.

By making Amsterdam Avenue a one-way street, for northbound traffic only, between West 178th and West 181st Streets, as indicated on Fig. 4, there will be no crossing of traffic lanes between these points. Any southbound local traffic on Amsterdam Avenue would be routed through Audubon Avenue for these three blocks.

This plan involves the acquisition of the entire block between West 178th Street, West 179th Street, Amsterdam Avenue, and Audubon Avenue, but it would provide a site sufficient for a future railroad terminal to serve tracks crossing both the George Washington and Harlem River Bridges and future connections between the former and a north and south route in Amsterdam Avenue.

As the route across The Bronx is planned as part of the metropolitan loop highway, previously mentioned, it should have convenient connections with Amsterdam Avenue for vehicles to and from points to the east. This is provided by two ramps in Highbridge Park, one of which would use part of that under construction at present (1933) as a connection for the tunnel in West 178th Street. The Washington Bridge at 181st Street would be left for local communication between The Bronx and Northern Manhattan. As this is the next highway bridge north of Macombs Dam Bridge at 155th Street, almost 1½ miles to the south, its full capacity will undoubtedly be required for that purpose.

The acquisition of only one block of property in Manhattan represents a small investment for such a major project as the proposed new Harlem River Bridge. The plan in Fig. 4 is only a suggestion as to how it might be

carried out. The writer believes that more detailed plans for a bridge at this site and a new highway across The Bronx, such as it involves, should now be advanced far enough so that the projected West 179th Street Tunnel and its connections can be fitted into them.

HUNLEY ABBOTT,[14] M. AM. SOC. C. E. (by letter).—A clear and interesting description of the approaches to the George Washington Bridge is presented by Mr. Evans. Modern high-speed motor traffic has changed completely the problem of bridge approaches compared to those of earlier days when slow-moving vehicles were the only ones to be considered. From this standpoint the design of these approaches is a fine, progressive piece of work.

The three chief principles laid down by Mr. Evans (no stopping of traffic, no crossing at grade, and no left turns) have been previously developed in the so-called "clover leaf" highway crossings, and will continue to be "fundamental laws" both for such crossings and for bridge approaches. The decentralization of traffic lanes everywhere up to the bridge itself is also a most important desiratum.

As a user of these bridge approaches, the writer's only criticism is that a motorist moving south on Riverside Drive en route to New Jersey must travel about half a mile south of the bridge and then double back an equal distance to get on the bridge. Presumably, this undesirable condition was made necessary by the difficult topography at this point, and because of excessive destruction of park property that might have been required by a more direct approach from the north. It is to be hoped, however, that some day this one-mile detour may be eliminated.

The bridge as a whole will stand out as a historic milestone in the development of bridge engineering. Its approaches will also stand out as an important milestone in the development of the science of rapid bridge transportation. For many years to come, engineers will use these approaches as a standard and a guide for similar development.

J. C. EVANS,[15] ESQ. (by letter).—The writer feels that the several discussions of his paper on the approaches of the George Washington Bridge have brought out a number of important points. He appreciates the emphasis placed upon a correct conception of the bridge approach and the basic principles underlying approach design in the discussion by Mr. Watson.

In discussing the relation of the facility and its approaches to the general scheme for the development of the highway system of New York and Its Environs, Mr Lewis, has developed an important phase of the subject. He has called attention to the lack of adequate provision for connections with trunk-line highways leading to the east or northeast on the New York side.

Undoubtedly, the bridge traffic would be greatly stimulated by adequate connections to arterial highways beyond Amsterdam Avenue, and the writer feels that the Regional Plan should be highly commended for its emphasis upon planning now for such connections. In the meantime, however, the projected construction of a new bridge across the Harlem River, which is

[14] Pres., Abbott, Merkt & Co., Inc., Engrs. and Archts., New York, N. Y.
[15] Terminal Engr., The Port of New York Authority, New York, N. Y.

beyond the jurisdiction of the Port Authority, is as yet so indefinite as to warrant no material change in the 178th Street Tunnel, now partly completed. It has been realized that the construction of the bridge, when definitely planned, would involve some major alterations at that point.

Mr. Abbott mentioned the disadvantage to the motorist moving south on Riverside Drive en route to New Jersey in having to travel a considerable distance south of the bridge in order to enter the approach. This condition is recognized, but was unavoidable except at great expense, not warranted by the possible benefits to a comparatively few motorists, due to the topography and interference with highly developed real estate. Under present conditions many motorists bound for New Jersey from the area north of the bridge utilize Broadway south of Dyckman Street, rather than Riverside Drive, because the Broadway route is somewhat shorter and has easier grades.

The writer appreciates the discussion by Mr. McClave. Eighteen months of operation of the bridge have indicated the soundness of the approach design. The New Jersey approaches have proved very satisfactory, except for the temporary condition which requires a sharp turn for Riverside Drive traffic at Northern Avenue, which condition was imposed by the necessity of adapting the initial stage of the approach to that of the ultimate approach.

The general layout of the toll facilities has also proved satisfactory. As might be anticipated, operating conditions have suggested certain minor modifications which can be incorporated to advantage in future designs.